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CONTENTS

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Editorial

This first issue of the REAAA Journal for 2007 contains six papers. These papers were awarded Katahira Awards at the 12th REAAA Conference held in Manila, Philippines, in November 2006. The Award is in honour of the late Dr Nobutaka Katahira, a former President of REAAA, who set up a trust fund to encourage the professional development of young engineers in the region. A judging panel was assigned to review eligible papers and make recommendations regarding the awards.

The first paper, by Yukihiro Tsukada, Waka Matsuda and Kenta Hamaya of the Ministry of Land, Infrastructure and Transport (MLIT), Japan, was awarded the Katahira Award for the best paper at the REAAA Conference. The paper describes a study conducted by the MLIT which examined the effects of flexible toll measures and the optimal layout of smart interchanges on traffic using the Japan expressway network. The analysis of the results obtained from demonstration projects suggested that the toll rate policy would be more efficient if it was set to specific time zones, such as commuting hours or night-time travel; appropriate time zones for policy implementation varied depending on the traffic characteristics; and greater efficiency would be achieved by selecting sections where policy implementation is likely to be particularly effective.

The second paper, by Seungjun Lee of the Korea Highway Corporation, was awarded the runner-up Katahira Award for outstanding paper at the Conference. In this study, an attempt was made to redefine the headway distribution at a freeway ramp merging area taking account of the effects of merging behaviour on freeway merging capacity calculations. Models for the calculation of merging capacity were developed which consider both freeway mainline and ramp flows. A sensitivity analysis was conducted to examine the impacts of different volume levels on freeway ramp merging capacity.

The next four papers were each awarded a Katahira Award for 'highly commended' paper at the REAAA Conference.

The Japan Highway Public Corporation has been responsible for the construction, operation and maintenance of the nation-wide toll expressways on behalf of the Japanese Government since 1956. In 2005 it was privatised, and three regional expressway companies (NEXCO) were formed. This paper describes the work undertaken by NEXCO to develop a pavement management system for the prediction of pavement performance, including life cycle costing (LCC). When the LCC models were applied to one section of the network, the durability of porous asphalt was about 1.2 times to 1.5 times higher than that of dense-graded asphalt, suggesting that the porous pavement is an excellent option for the rehabilitation of pavements about ten years of age or older. Ride quality is becoming an increasingly important issue on Japanese expressways. For this reason, NEXCO has also been investigating the applicability of the International Roughness Index (IRI) to assess the ride quality of its network and to assist in the setting of limits for its safe and efficient operation.

In 1999, the Government of Malaysia regulated passenger van services in Bangkok to assist in the provision of safe public transport to commuters and to reduce competition between passenger vans and conventional buses. However, a large number of illegal vans still operate along the licensed

routes. The objectives of the study described in the fourth paper were to investigate the level-of-service attributes of public transit users in general, and illegal passenger van users in particular, and to use these findings in the planning of future services. The three main reasons why respondents preferred to use the passenger van service were related to time, convenience, and comfort. Most respondents were unaware that the vans they were using were operating illegally because their characteristics and services were similar to the legal passenger vans.

The next paper describes work currently being conducted by ARRB Research to improve knowledge of the effects of axle load and tyre type on the deterioration of unbound granular pavements surfaced with a thin bituminous seal, the most common pavement type in Australasia. The two specific objectives of the research were to determine the applicability of the 'fourth power law' to these types of pavements and to assess the relative damaging effects of wide single and steer axle tyres compared to dual tyres for Australasian pavement types.

Serious traffic congestion at intersections has become a critical issue in Japan. The final paper describes a system developed by the Japan Bridge Association for the rapid construction of over-bridge crossings to minimise traffic restrictions, reduce construction costs, and minimise environmental impacts. An outline of the rapid construction method is presented, together with two case studies.

The Editorial Panel continues to seek papers and technical notes for publication in the Journal. The membership of the Editorial Panel follows. REAAA members interested in submitting a paper should seek advice from the appropriate member(s) of the Editorial Panel. The Panel is striving to publish at least one paper from each Chapter or region each year.

Kieran Sharp
Chairman REAAA Technical Committee

MEMBERSHIP OF EDITORIAL PANEL: REAAA JOURNAL

The Effect of Flexible Toll Measures and Optimal Layout of Smart Interchanges on the Japan Expressway Network*

Yukihiro Tsukada, Waka Matsuda and Kenta Hamaya

Ministry of Land, Infrastructure and Transport, Japan

ABSTRACT

The use of expressways in Japan is lower than that in Europe and North America, with potential users preferring to drive on roads running parallel to the expressways. This has resulted in problems, related to congestion, safety and the deterioration of the roadside environment on these roads. The Ministry of Land, Infrastructure and Transport (MLIT) is developing policies to address these problems and encourage the use of expressways. This paper describes a study conducted by the MLIT which examined the effects of flexible toll measures and the optimal layout of smart interchanges on traffic using the Japan expressway network. The analysis of the results obtained from demonstration projects on toll road charges showed that the toll rate policy would be more efficient if it was set to specific time zones, such as commuting hours or night-time travel; the appropriate time zones for policy implementation varied depending on the traffic characteristics of the area; and greater efficiency would be achieved by selecting sections where policy implementation is likely to be particularly effective. The results of the demonstration project on Smart IC showed that there were two types of usage characteristics and that these usage characteristics differed depending on the local characteristics. The study also suggested that the same plan should not be implemented throughout the country; if local traffic characteristics or traffic problems unique to each area are taken into consideration, then strategies tailored to solving those specific problems will be more effective.

1. INTRODUCTION

The use of expressways in Japan is lower than that in Europe and North America, with potential users preferring to drive on roads running parallel to the expressways. This has resulted in problems, related to congestion, safety and the deterioration of the roadside environment on these roads. The Ministry of Land, Infrastructure and Transport (MLIT) is developing the following three policies to address these problems and encourage the use of expressways.

- ❑ Elimination of missing links: faster elimination of the missing links in the expressway network to assist the road network to deliver the intended speed. A case study in Shikoku showed that opening a 9 km long section, which had been the only gap in the entire 120 km long route, resulted in an increase in traffic volume of 30%.
- ❑ Flexible charge measure: flexible toll road charges to help reduce the tolls currently applied on Japanese expressways.
- ❑ The construction of additional 'smart interchanges' (ICs) to shorten the long interval between them (about twice as long as the USA or Europe). The provision of additional ICs on the Tomei Expressway resulted in an increase in traffic volume on the expressway and mitigated congestion on the neighbouring ICs and on the national road running parallel with the Expressway.

This paper presents details of the study and its effects on traffic flow on interchanges in Japan.

* This paper was awarded the Katahira Award for the best paper at the REAAA Conference in Manila in November 2007.

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2. DESCRIPTION OF THE STUDY

In this research, the focus was on demonstration projects of toll road charges and Smart IC conducted in various parts of Japan, and the use of the results to identify those traffic characteristics that reacted sensitively to the toll road charges or the construction of the additional ICs. Toll road charge measures that ensured greater efficiency and effectiveness were also developed.

2.1 Analysis of Toll Measures

2.1.1 Outline of Demonstration Projects on Toll Road Charge

The Road Bureau of the MLIT launched demonstration projects on toll road charges in 2003 in an attempt to reduce the high tolls, which were discouraging drivers from using the expressways (see Tables 1 and 2). In addition, the Bureau is currently running a time zone discount plan (during the night, commuting time, etc.) for cars equipped with ETC based on the results of the demonstration projects (see Table 3).

Table 1: Demonstration projects on toll road charges (2004)

Problem \ Area	Nationwide or Area-Wide	Suburbs of Major Cities	Local Core Cities	Local Cities and Others
congestion		2	11	3
roadside environment		1	2	1
tourism	1			1

Table 2: Demonstration projects on toll road charges (2005)

Problem	Nationwide and Wide-Area	Suburbs of Major Cities	Local Core Cities	Local Cities and Others
congestion		6	17	12
roadside environment		2	1	1
tourism	1			2 (including one re-listed)

Table 3: Time zone discount on expressways

Commute Time Discount (Local Area) (commenced 11 January 2005)	Early Morning and Night time Discount (Large Cities) (commenced 11 January 2005)	Midnight Discount (Entire Nation) (commenced 11 November 2004)
o for effective use of expressways with spare capacity (excluding large cities) o 50% discount during morning and evening rush hours every day (0600–0900, 1700–2000) o only if the distance travelled is less than 100 km	o correction of traffic volume imbalance between daytime and night-time for expressways in major cities o 50% discount for early morning and night time hours in major cities (2200–1800) o only if the distance travelled is less than 100 km	o promotion of night time use of expressways o 30% discount during midnight hours (0000-0400)

Verification of Efficient Time Discounting

Table 4 shows the calculated toll rate elasticity value during the course of a day for a number of local cities. The toll rate elasticity is an index of the sensitivity of traffic volume to a change in the toll. It is calculated by:

$$\text{Toll Rate Elasticity Value} = -\frac{\dfrac{Q'-(Q+Q')/2}{(Q+Q')/2}}{\dfrac{P'-(P+P')/2}{(P+P')/2}} \qquad (1)$$

where Q = traffic volume before the experiment
Q' = traffic volume during the experiment
P = toll before the experiment
P' = toll during the experiment.

Table 4: Toll Rate Elasticity Value by Time Period

	All day	Morning (0700–0900)	Day (0900–1700)	Evening (1700-2100)	Night (2200–0600)
Aomori	0.38	0.47	0.36	0.38	0.07
Hitachi	0.69	0.54	0.54	1.03	0.83
Niigata	0.76	0.81	0.60	0.96	0.53
Itoigawa	1.36	1.08	1.38	1.51	2.83
Toyama	1.11	1.11	1.04	1.22	1.27
Kanazawa	0.88	0.74	0.82	1.06	1.24
Okayama	0.55	0.43	0.45	0.82	0.84
Shimane	0.50	0.46	0.49	0.50	0.57
Hiroshima	0.44	0.65	0.35	0.38	0.57
Kure	0.37	0.39	0.35	0.41	0.24
Average	0.55	0.58	0.48	0.62	0.43

The shaded cells indicate those toll rate elasticity values which were equal to, or greater than, the value for the entire day. It can be seen that the values were higher during the commuting hours in the morning or evening than the entire day and particularly higher in the evening.

Generally, the traffic volumes, and hence profit, tended to increase as a result of offering discount tolls for specific time zones rather than for the whole day. The degree of effectiveness of discount time zones varies depending on the area. For example, if an area has many commuters, then offering a discount in the morning and evening peak time zones will be effective. If there is more through traffic, then a discount for night time use is effective.

Time zone discount rate plans should be developed which are tailored to local traffic conditions. It is also important to understand the problems caused by specific local characteristics and to set discount rates in the time zones that appropriately address these problems. The implementation of a flexible toll rate tailored for local conditions would be highly effective.

Verification of Efficient Section Setting

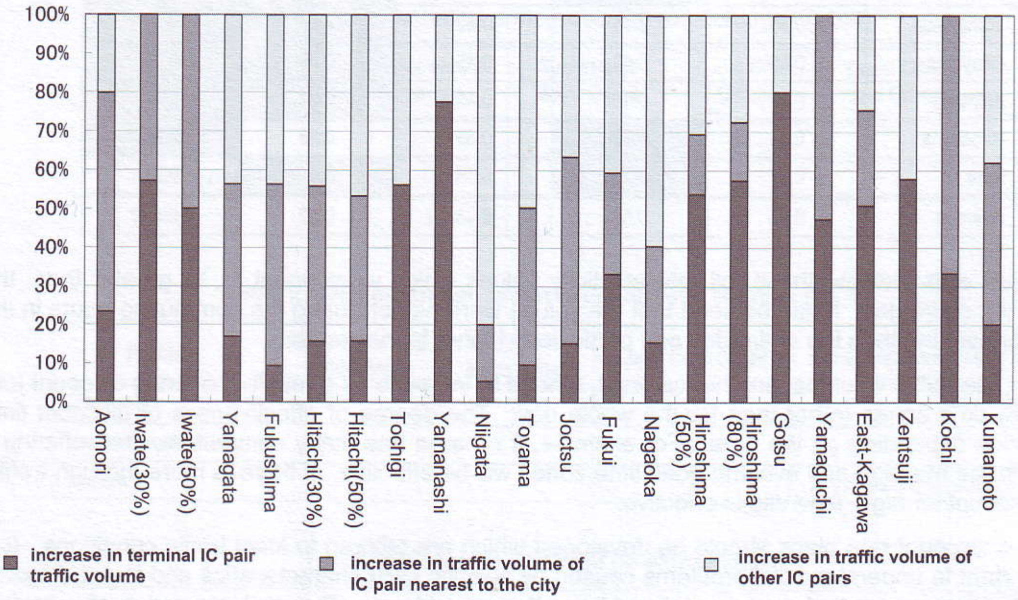
The demonstration projects conducted by MLIT involved a 50% discount in toll charges on four road sections during 2003 and 2004. Table 5 shows the changes in traffic volume along these sections, and the toll rate elasticity value, in 2003 and 2004. The results for each section showed a tendency of a declining increase in expressway traffic volume. This was because the frequency of the IC pair used was dispersed over many neighbouring ICs due to the increase in the number of IC pairs used in the discount section. The toll rate elasticity value also decreased, indicating a possible reduction in profitability. These results indicate that discount toll rate setting would result in greater efficiency if the section is not too long and that there are not problems, such as traffic congestion, on parallel general roads.

2.1.2 Impacts of Local Characteristics

Figure. 1 shows the increase in traffic volumes on the expressways in the demonstration projects according to the increase in terminal IC pair traffic volume (assumed to be through traffic), the increase in traffic volume of the IC pair closest to the city area (assumed to be commuting and service traffic), and the increase in traffic volume of other IC pairs. The results varied according to location.

Table 5: Changes in Traffic Volume on Expressway Sections and Toll Rate Elasticity Value

	2003		2004	
	Changes in expressway traffic volume	Toll Rate Elasticity Value	Changes in expressway traffic volume	Toll Rate Elasticity Value
Hitachi	1.7 times (4,455 –> 7,471 vehicles/day)	0.69	1.6 times (7,087 –> 11,637 vehicles/day)	0.71
Niigata	1.7 times (2,037 –> 2,771 vehicles/day)	0.76	1.2 times (15,003 –> 18,463 vehicles/day)	0.33
Itoigawa	2.6 times (174 –> 446 vehicles/day)	1.36	1.8 times (2,398 –> 4,222 vehicles/day)	1.04
Toyama	2.2 times (1,810 –> 3,929 vehicles/day)	1.11	1.5 times (8,532 –> 12,788 vehicles/day)	0.67



* For ICs nearest to the urban area of a relatively large city, one such IC is counted as the subject for this experiment.

* No ICs nearest to the urban area are set when there are no relatively large urban areas except for the terminal IC.

Figure 1: Classification by type of characteristics of traffic volume that increased during the experiment

The demonstration areas were classified according to traffic characteristics. The local characteristics for each type, the major demonstration results and their trends are listed in Table 6. Cities having a population of about 50,000 were classified as 'through traffic', while cities having a population over 100,000 were classified as 'commuting and service' or 'mixture' type.

In terms of accessibility to ICs, if the access distance from a general national road to a terminal IC was very short (0-0.7 km) then this was classified as 'through traffic'. If accessibility was poor (i.e. distance of 1.2-7.5 km from the IC nearest to the city area to the urban area or the general national road), then this was classified as 'commuting or service' or 'mixture' because the expressway route went around the city area (excluding the demonstration project in Aomori).

The most significant effects of the demonstration projects were:

- ❑ an increase in travel speed or a reduction in noise levels on the general national roads, mainly in the daytime or during the night for the 'through traffic' type
- ❑ the mitigation of congestion on general roads, mainly during the morning and evening rush hours for the 'commuting and service traffic' type

- mitigation of congestion during the morning and evening rush hours and a part reduction in noise levels during the night for the 'mixture' type.

If local traffic problems are mainly caused by through traffic on general roads and if this traffic needs to be diverted to the expressways, then ICs with good accessibility should be constructed. If attractive cities are located in the area, commuting and service traffic tends to shift from the general roads to the expressway despite poorer accessibility. In other words, if access from the expressway to the urban area is improved, then there will be a greater shift in traffic from the general roads to the expressway. Regardless, it is important that the local characteristics in the area where the toll rate plan is to be implemented are assessed.

Table 6: Local characteristics by type of traffic characteristics and effects trends

Type of traffic characteristics	Existence of attractive city	Access to IC	Major project effects and manifestation of effects
Through traffic increase of at least 50% in traffic volume of the terminal IC pair but increase of less than 20% in traffic volume of the IC pair nearest to the city (Tochigi, Yamanashi, Gotsu, East-Kagawa, Zentsuji)*	None no large city in project area	Good access from national road to terminal IC good compared with internal section IC	- average Elasticity Value of 0.96** - mainly effective in reducing congestion and noise in daytime and night time
Commuting/business traffic increment of at least 40% in traffic volume of the IC pair nearest to the city (Yamagata, Fukushima, Hitachi, Toyama, Itoigawa)	Exist large cities exist in project area	Bad access from IC nearest to the city (or national road) not good	- average Elasticity Value of 0.71** - mainly effective in reducing congestion during morning and evening peak hours
Mixture increase in traffic volume of terminal IC pair and that of IC pair nearest to the city both at least 20% (Aomori, Iwate, Fukui, Yamaguchi, Kochi, Kumamoto)	Exist large cities exist in project area	Good/Bad access to terminal IC pair relatively good, but access to IC nearest the city not so good	- average Elasticity Value of 0.87** - effective in reducing congestion during morning and evening peak hours and partly reducing noise levels during the night
Distributed increase in traffic volume of other IC pairs at least 50% (Niigata and Nagaoka)			

* Average Elasticity Value: weighted increase in traffic volume.

** Hiroshima was excluded because the project was designed for large vehicles.

2.2 Analysis of Measure of Number of ICs (Smart IC)

2.2.1 Overview of the Demonstration Projects on Smart IC

One factor that makes expressways less user-friendly is that the interval between ICs is often too long. To shorten the interval between ICs, the demonstration project on Smart IC for connecting to SA (service areas) or PA (parking areas) was launched in 2004. The project is currently under way at 32 locations. These newly-constructed ICs are called 'Smart IC' and designed especially for cars equipped with ETC. The cost of constructing and managing Smart IC is about 30 to 50% less than that of conventional ICs, so a lower cost would be incurred when newly constructing Smart ICs (see Figure 2).

This following section analyses the usage characteristics of these additional ICs and their effects, and identifies factors that affect the number of cars that use the new ICs.

Image of the Smart IC connecting to SA and PA

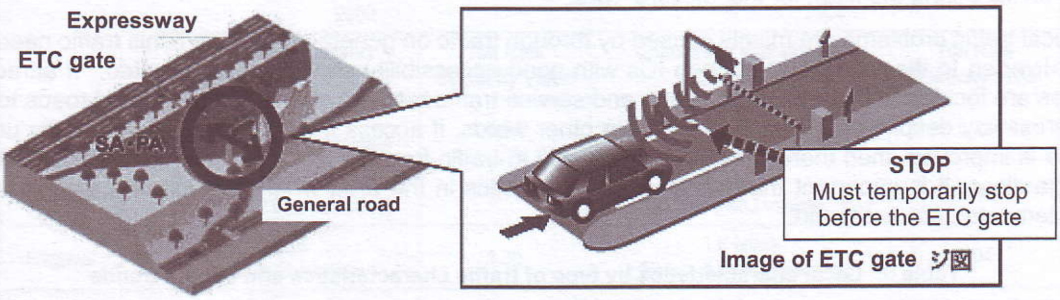


Figure 2: Image of demonstration projects on the Smart IC connecting to SA/PA

2.2.2 Usage Characteristics of Smart IC

The usage characteristics of the additional Smart ICs may be categorized into two patterns: (a) 'induced use', or users who have not often used expressways before this demonstration project, and (b) 'shifted use', where regular users of the expressway change the ICs that they use.

The benefits associated with the 'induced use' include mitigation of congestion on general roads, improvement to the roadside environment, increased traffic safety through the normalisation of community roads, revitalisation of the community through enhanced mobility, and higher profits for the expressway company. The expected benefits of the 'shifted use' scenario include the elimination of concentrated use of certain ICs, more appropriate dispersal of traffic, and smoother traffic flow. These benefits will be fully gained when there is traffic congestion on conventional ICs adjacent to the Smart ICs.

The number of cars induced to use the expressway for the first time and the number of cars shifting to the newly-installed ICs were calculated using the following equations:

$$\text{induced car count} = Q'_S - Q_S\left(\frac{Q'_A}{Q_A}\right) \tag{2}$$

$$\text{shifted car count} = (\text{number of cars using Smart IC}) - (\text{induced car count}) \tag{3}$$

where Q_S = number of cars using ICs adjacent to Smart IC before the demonstration project

Q'_S = number of cars using Smart ICs and their adjacent ICs during the demonstration project

Q_A = number of cars using the expressway along the entire length before the demonstration project (section excluding Smart ICs and their adjacent ICs)

Q'_A = number of cars using the expressway along the entire length during the demonstration project (section excluding Smart ICs and their adjacent ICs)

Note that all the traffic induced to use the expressway following the installation of the Smart ICs was assumed to always use Smart ICs, and the number of cars using Smart ICs was set as the upper limit.

Figure 3 shows the calculated number of induced and shifted cars for each Smart IC. Smart ICs located within a radius of 15 km from a city larger than the population of the preferential city (200,000 or more) (herein referred to as the 'central city') were categorised as 'near-city' types, whilst the other Smart ICs were classified as 'locally-located'. It can be seen that there were 'induced' cars in both the near-city and locally-located types, but there were more 'induced' cars in the near-city locations.

'Induced' cars occurred regardless of the location of the additional Smart IC, which means that residents living near those ICs commenced using the expressway because of the shorter time taken to reach the new ICs. This pattern of use is called 'resident use'.

There were more 'shifted' cars in the near-city type because ICs closest to the expressway near a larger city are subject to concentrated traffic during peak hours. As a result, drivers shift to the Smart ICs to avoid the congestion. This pattern of use is called 'congestion-avoiding use'.

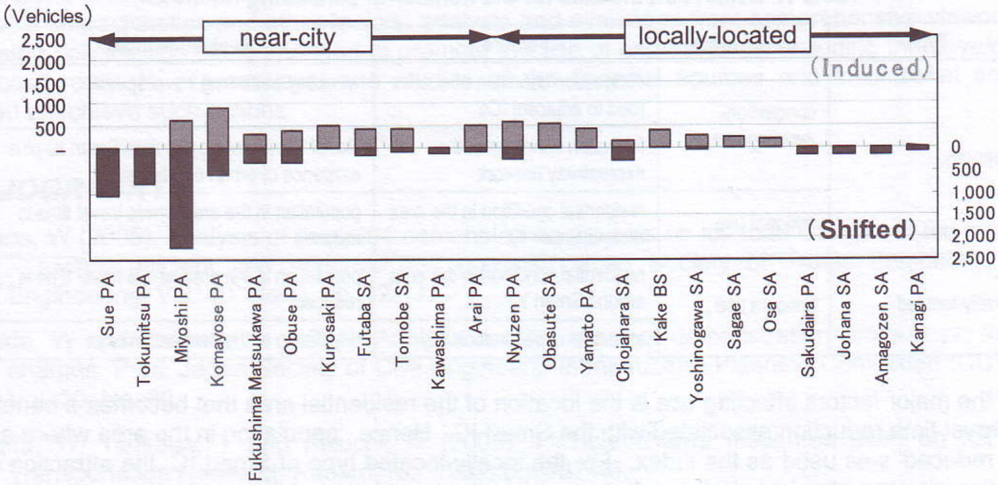
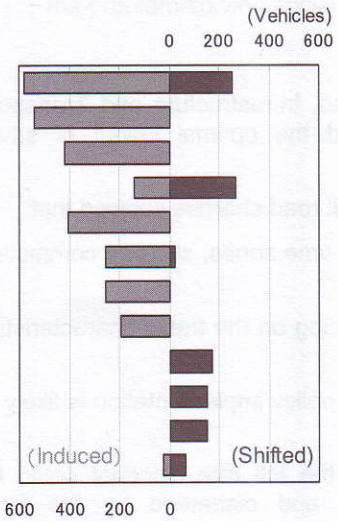


Figure 3: Breakdown of number of induced and shifted vehicles

Figure 4 shows the relationship between the main destination IC of vehicles using Smart IC with the induced and shifted car numbers for the locally-located type. The results show that, when there were many induced cars, the top IC pair was located within a radius of 60 km from the central city. This result suggests that the increased use of Smart IC reflects the attraction of the city or the strong links between the city and the areas near the Smart IC.



Smart IC	Top IC Pair		Main City within 60km radius
	IC	Distance(k	
Nyuzen PA	Toyama	43.8	Toyama
Obasute SA	Matsumoto	37.8	Matsumoto、Nagano
Yamato PA	Nagaoka	46.2	Nagaoka
ChojaharaSA	Izumi	34.9	Sendai
Kake BS	Hiroshima-Kita	15.1	Hiroshima
Yoshinogawa SA	Aizumi	59.4	Tokushima
Sagae PA	Yamagata-Zao	20.3	Yamagata
Osa SA	Niimi	12.3	None
Sakudaira PA	Nerima	146.9	Nagano、Takasaki
Johana SA	Gokayama	12.5	Toyama, Kanazawa
Amagozen SA	Kanazawa-Nishi	35.7	Kanazawa, Fukui
Kanagi PA	Hamada	9.5	None

Figure 4: Relationship between number of induced cars and main cities (locally-located)

2.2.3 Analysis of Factors Affecting the Number of Cars Using Smart IC

The results of the analysis of the usage characteristics were used to identify the factors that affected the number of cars using Smart IC. The evaluation indexes that represent those factors are listed in Table 7.

One of the main factors that affected congestion-avoiding use was congestion on the access roads to the adjacent ICs, for which the weekday travel speed during congestion, quoted from the Road Traffic Census, was used as the evaluation index. If there are many ICs closer to the city than the Smart IC, and if traffic is already dispersed, or if an urban expressway is constructed in the city area and the urban expressway is put to full use, then, logically, traffic congestion will be reduced. Likewise, the congestion-avoiding factor provided by Smart IC will lose its effectiveness if the expressway network is already fully developed in the area. Therefore, the 'number of ICs' and 'urban expressways' were used as optional indexes.

Table 7: Evaluation indexes for the number of cars using Smart IC

Type	Usage mode	Factors	Evaluation index
near-city	congestion-avoiding use	congestion condition of access road to adjacent ICs	weekday peak-time travel speed
		formation condition of the expressway network	No. of ICs nearer to city than Smart IC plus existence of urban expressway
	resident use	residential condition in the area around Smart IC	population in the area where travel time is reduced
locally-located	resident use	residential condition in the area around Smart IC	population in the area where travel time is reduced
		attraction of destination city	population of the destination city

One of the major factors affecting use is the location of the residential area that becomes a beneficiary of the travel time reduction associated with the Smart IC. Hence, 'population in the area where access time is reduced' was used as the index. For the locally-located type of Smart IC, the attraction of the destination city was also included as a factor, and so the 'population of the destination central city' was added as an index.

Using these indexes, the number of cars using Smart IC was analysed and the results are shown in Figures 5 and 6. For the near-city type, the number of cars using Smart IC tended to be larger for both the population in the area where the travel time was reduced and the weekday peak-time travel speed was high. For locations where the expressway network was considered to be good, the number of cars tended to be smaller than in other locations having the same index scores. For the locally-located type, the number of cars tended to be greater if the scores for the population in the reduced-travel-time area and the population of the destination central city were both high.

3. CONCLUSIONS

This paper has described a study conducted by the Ministry of Land, Infrastructure and Transport, Japan which examined the effects of flexible toll measures and the optimal layout of smart interchanges on traffic using the Japan expressway network.

The analysis of the results obtained from demonstration projects on toll road charges showed that:

- the toll rate policy would be more efficient if it was set to specific time zones, such as commuting hours or night-time travel
- the appropriate time zones for policy implementation vary depending on the traffic characteristics of the area
- greater efficiency would be achieved by selecting sections where policy implementation is likely to be particularly effective.

The study also showed that the traffic characteristics reacting to the toll rate discount could be categorised into through traffic, commuting and service traffic, and dispersed by the local characteristics. Each type is different in that some have attractive cities in the local area, or the condition of access from the expressway to a general road or urban area is different.

The results of the demonstration project on Smart IC showed that there were two types of usage characteristics, 'induction' and 'shifting', and that these usage characteristics differed depending on the local characteristics. Factors that affect the number of cars using Smart IC were clarified based on an analysis using indexes representing the traffic condition and population distribution in the surrounding area.

This study showed that traffic characteristics and traffic-related problems vary according to the area and that the effects of the policies vary accordingly. To ensure the more effective use of expressways, the same plan should not be implemented throughout the country; if local traffic characteristics or traffic problems unique to each area are taken into consideration, then strategies tailored to solving those specific problems will be more effective.

Future tasks aimed at improving these policies include the need for further analysis of the relationship with local characteristics and other factors, analysis and evaluation from comprehensive viewpoints of all related policies and plans intended to promote the use of expressways, including the development of access roads to expressways, and studies of the financial sources and institutional schemes needed to achieve such actions.

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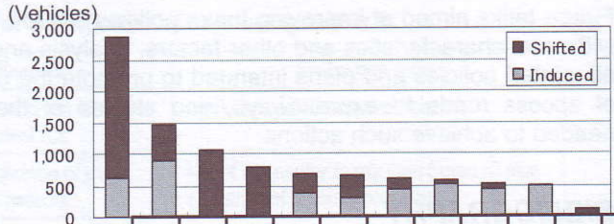
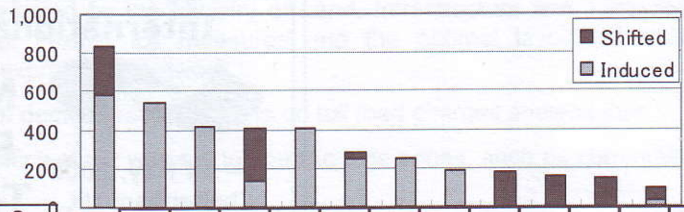


Condition of Expressway Network	
M	Two or more ICs nearer to the main city than the Smart
U	Urban Expressway exsists

Rank	A	B	C
Week-day Peak-time Travel Speed	less than 20km/h	20 - 30km/h	30km/h or more
Population in the Reduced Travel Time	50,000 or more	10,000 - 50,000	less than 10,000

		Miyoshi PA	Komayose PA	Sue PA	Tokumitsu PA	Obuse PA	Fukushima Matsukawa PA	Futaba PA	Arai PA	Tomobe SA	Kurosaki PA	Kawashima PA
Congestion Avoiding Index	Week-day Peak-time Travel Speed	B	A	B	B	B	C	B	C	B	A	C
	Condition Of Expressway Network	-	-	U	-	-	-	-	M	M	M	-
Resident Use Index	Population in the Reduced Travel Time	A	A	A	B	B	B	B	B	C	B	C

Figure 5: Factors affecting the number of cars using Smart IC (near-city)



Rank	A	B	C
Population in the Reduced Travel Time	50,000 or more	10,000 - 50,000	less than 10,000
Population of Destination Main City	1million or more	200,000 - 1million	None

		Nyuzen PA	Obasute SA	Yamato PA	Chojahara SA	Kake BS	YoshinogawaSA	Sagae SA	Osa SA	Sakudaira PA	Johana SA	Amagozen SA	Kanagi PA
Resident Use Index	Population in the Reduced Travel Time	C	A	B	C	C	B	B	C	C	C	C	C
	Population of Destination Main City	B	B	B	A	A	B	B	C	B	B	B	C

Figure 6: Factors affecting the number of cars using Smart IC (locally-located)

Mathematical Modelling of Freeway Ramp Merging Capacity Analysis Based on Revised Headway Distribution Model*

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ABSTRACT

The determination of the merging capacity of a freeway ramp section is important when the quality of traffic flow along a freeway system is being assessed. Whilst merging capacity of a freeway ramp merging area can be calculated by analysing headway distributions on a freeway upstream of freeway ramps, the models tend to overestimate capacity because the headway distribution on the freeway changes prior to its arriving at the ramp segment. In the study described in this paper, an attempt was made to redefine the headway distribution at a freeway ramp merging area by taking account of the effects of merging behaviour on freeway merging capacity calculations. Models for the calculation of merging capacity were developed which consider both freeway mainline and ramp flows and a sensitivity analysis was conducted to examine the impacts of different volume levels on freeway ramp merging capacity. Further research is required to develop a simpler method of analysis for various volume levels. Recalibration of the input parameters for the headway distribution model is also required to increase the practicality of the developed model.

1. INTRODUCTION

The merging capacity of a freeway ramp is important in the design, operation and planning of a freeway system, because the level of service of the freeway system is usually dependent on freeway ramp merging capacity. In practice the US Highway Capacity Manual is normally used to determine freeway ramp capacity. However, the use of the Manual to determine merging capacity has some limitations related to the assumptions made when describing the headway distributions of freeway mainline flow.

In the study described in this paper, a review was undertaken to determine what was the most relevant headway distribution for calculating the merging capacity of a freeway ramp. During this review, it was noted that the negative exponential distribution is normally used for describing headway distribution. However, because of vehicle interactions at freeway ramp areas, it is suggested that a headway distribution different from the negative exponential distribution could be adopted because applying an inaccurate time headway distribution will result in an overestimation of merging capacity on freeway ramp sections.

The following two steps were undertaken in this study:

- ❑ comparison of headway distributions between freeway sections and ramp areas using statistical analysis of collected headway data
- ❑ calibration of parameters for the selected headway distribution.

Mathematical models for calculating freeway ramp merging capacity were also developed, with a gap acceptance procedure for an individual vehicle forming the basic structure of this model. A sensitivity analysis was conducted to check how freeway ramp merging capacity varies with changing traffic flow volume levels.

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* This paper was awarded the runner-up Katahira Award for outstanding paper at the REAAA Conference in Manila in November 2007.

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2. LITERATURE REVIEW

2.1 Headway Distributions

Table 1 presents the Pearson Type III distribution models most widely used for modelling traffic flows on freeways. Depending upon volume levels, different forms of headway distributions are used as follows:

- negative exponential distribution for random headway state
- normal distribution for constant headway state
- one of the following three approaches for the intermediate headway state:
  - generalized mathematical model
  - composite model
  - other.

Table 1: Pearson Type III Distribution Models

Distribution Family	Estimated $\hat{K}$	Calculated λ	Probability Density Function, f(t)
Pearson Type III (K, α)	$\frac{\bar{t}-a}{s}$	$\frac{K}{\bar{t}-\alpha}$	$\frac{\lambda}{\tau(K)} \cdot [\lambda(t-a)]^{K-1} \cdot e^{-\lambda(t-a)}$
Gamma (K, α = 0)	$\frac{\bar{t}}{s}$	$\frac{K}{\bar{t}}$	$\frac{\lambda}{\tau(K)} \cdot [\lambda(t-a)]^{K-1} \cdot e^{-\lambda(t-a)}$
Erlang (K=1, 2, 3, ..., α = 0)	$\frac{\bar{t}}{s}$	$\frac{K}{\bar{t}}$	$\frac{\lambda}{\tau(K)} \cdot [\lambda(t-a)]^{K-1} \cdot e^{-\lambda(t-a)}$
Negative exponential (K = 1, α = 0)	$\frac{\bar{t}}{s}$	$\frac{1}{\bar{t}}$	$\frac{\lambda}{\tau(K)} \cdot [\lambda(t-a)]^{K-1} \cdot e^{-\lambda(t-a)}$
Shifted negative exponential (K = 1, α > 0)	$\frac{\bar{t}-a}{s}$	$\frac{1}{\bar{t}-\alpha}$	$\frac{\lambda}{\tau(K)} \cdot [\lambda(t-a)]^{K-1} \cdot e^{-\lambda(t-a)}$

2.2 Freeway Ramp Merging Capacity

The literature review revealed that the gap acceptance modelling approach is often used to determine merging capacity at freeway ramp areas (see eqn (1)). Multiple entries of *n* vehicles into a freeway merging area are modelled in this particular case.

$$P\{T_C + (n-1)t^* < t < T_C + nt^*\} = \int_{T_C+(n-1)t^*}^{T_C+nt^*} f(t)dt, \quad n \geq 1 \tag{1}$$

where T_c = critical time gap (seconds)

t^* = time gap to permit multiple entry (seconds).

Eqn (1) leads to eqn (2), the total number of ramp merging vehicles. This can be considered as the merging capacity.

$$\therefore Q_{MAX} = Q_m \cdot \sum_{n=1}^{\infty} P\{T_C + (n-1)t^* < t < T_C + nt^*\} \times n \tag{2}$$

where Q_m = ramp volumes entering into freeway main line flow (veh/h).

When the negative exponential distribution shown in eqn (3) is applied, eqn (4) can be obtained.

$$f(t) = \lambda \cdot e^{-\lambda t}, \ \lambda = \frac{1}{3600/Q_m} = \frac{1}{\bar{t}} \tag{3}$$

$$Q_{MAX} = Q_m \cdot \frac{e^{-\lambda\tau_c}}{1 - e^{-\lambda t^*}} \tag{4}$$

3. NEW APPROACH FOR CALCULATING FREEWAY RAMP MERGING CAPACITY

In this study it was assumed that the headway distribution at a freeway ramp area was different from that in the freeway basic section. In terms of the general shape of the proposed headway distribution, some more peaking characteristics and skewness would be expected to the right side of the graph shown in Figure 1. The modelling of the new freeway ramp merging capacity involves two tasks: calibrate the parameters in the headway distribution formula and, using the calibrated formula, develop a model for determining the number of merging vehicles. Rather then using the approach recommended in the US Highway Capacity Manual, this approach produces a set of variable merging capacities according to the many combinations of freeway main line and ramp volumes. This is understandable, because headway distributions would be different depending on downstream ramp volumes and this would result in a different merging capacity. After the converging process involving freeway mainline volumes and ramp volumes, one final ramp merging capacity can be obtained.

3.1 Headway Distribution at Freeway Ramp Merging Areas

Currently negative exponential distributions such as curve (1) in Figure 1 are often applied to freeway basic sections and used in developing ramp merging capacity. However, it is argued that curve (2) should be used to analyse ramp merging capacity because curve (1) does not reflect the impacts of freeway ramp vehicles on vehicle manoeuvres on the freeway mainline. These impacts include speed reduction, lane changes and changes in headway distribution. This study only focussed on changes to headway distribution.

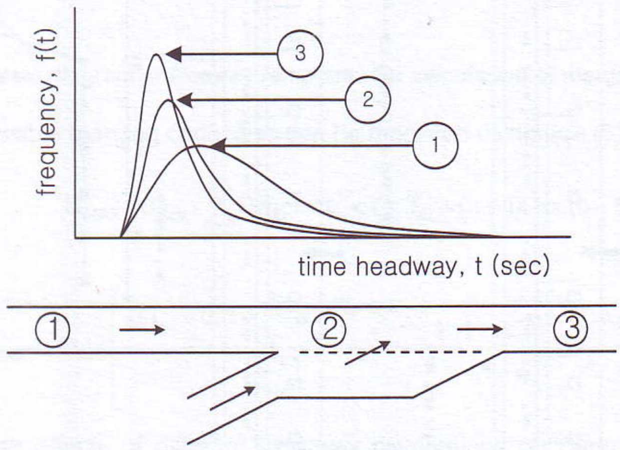

Figure 1: A set of different shapes of headway distributions for freeway ramp merging areas

In order to investigate the changed form of the headway distribution, it was considered that the shape of the time headway distribution for the freeway ramp merging area should be similar to one combining curve (1) and curve (3) in Figure 1. Figure 2 shows how the time headway distribution on the basic section varies in the merging area due to changes in the merging flow. It can also be seen from that merging opportunity decreases from 8 veh/minute (480 veh/h) to 7 veh/minute (420 veh/h).

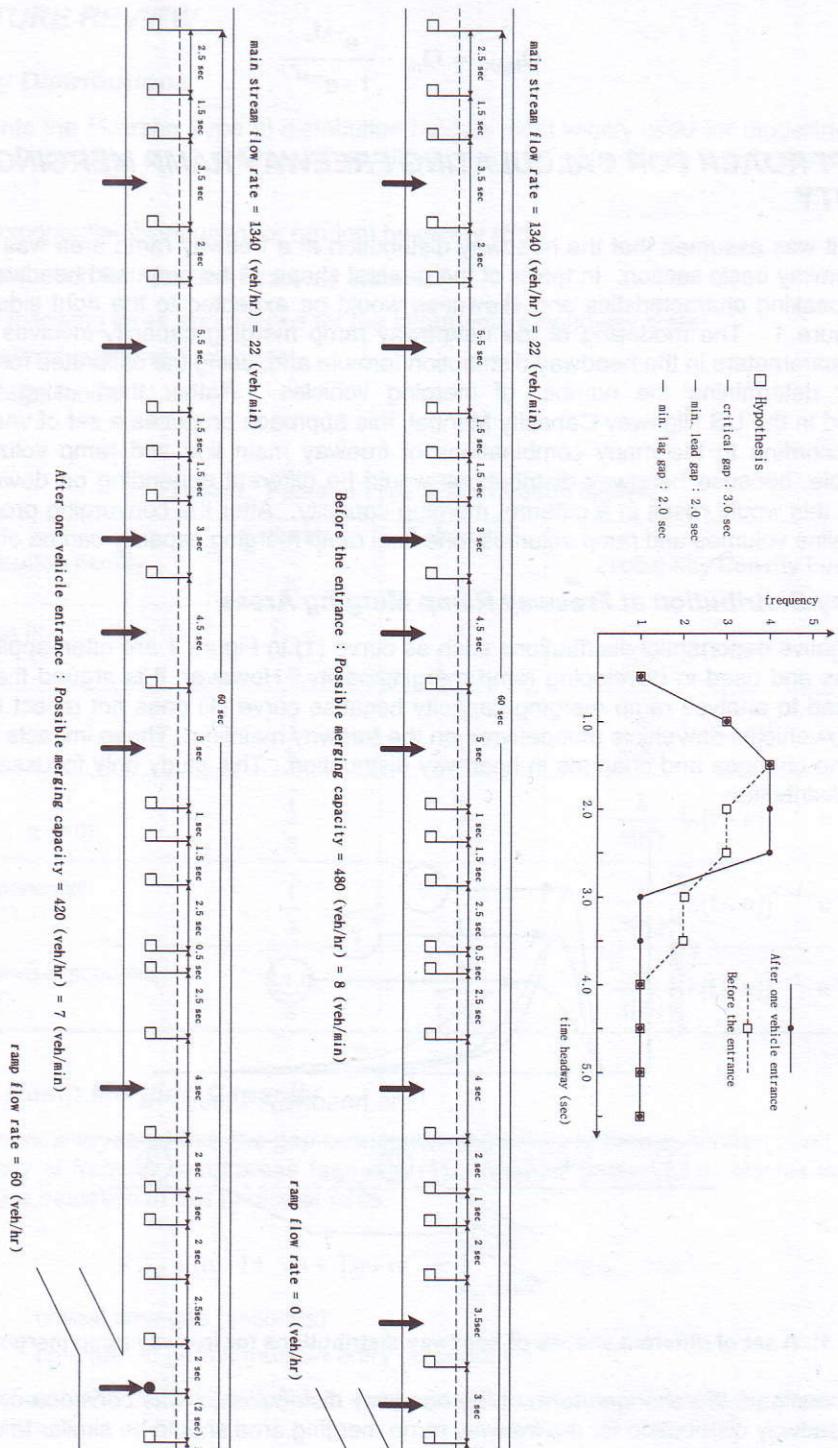

Figure 2: Example of variation in time headway distribution and merging opportunity in merging area

In this study, it was decided to use the Pearson Type III distribution (K = 2, α = 0.5) as shown in eqn (5) for the headway distribution on the freeway ramp merging area. The values of K and α were adopted tentatively after considering the fact that, when the parameter (K) is not an integer, the Pearson Type III distribution turns into an incomplete gamma function and the solution gets very complicated.

$$f(t) = \lambda^2 \cdot (t - 0.5) \cdot e^{-\lambda(t-0.5)}, \quad \lambda = \frac{2}{\dfrac{3600}{Q_m + W \cdot Q_r} - 0.5}, \quad W = \frac{Q_m}{3600} \tag{5}$$

where Q_m is the shoulder-lane flow (veh/h) and Q_r is the on-ramp flow (veh/h).

In addition, a parameter (W) was introduced to account for the effects of ramp merging vehicles on main line flows because it would be expected that these effects would be large with high volumes and vice versa. This also made it easier to address the problem of repeated calibrations of headway distributions for each level of freeway mainline and ramp merging volumes.

3.2 Ramp Merging Capacity Calculation

Figure 3 shows a typical freeway ramp area and merging vehicles. This diagram was important in the development of the ramp merging capacity calculation procedure reported in this paper.

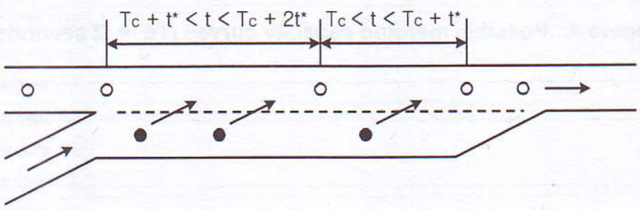

Figure 3: Basic diagram of freeway ramp area for calculation of merging capacity

Looking at Figure 3, possible merging capacities can be modelled using eqn (6).

$$Q_{MAX} = Q_m \cdot \sum_{n=1}^{\infty} P\{T_C + nt^* < t < T_C + (n+1)t^*\} \times (n+1)$$

$$Q_{MAX} = Q_m \cdot \left[\frac{\lambda(T_C - 0.5 + \frac{1}{\lambda} \cdot e^{-\lambda(T_C - 0.5)}}{1 - e^{-\lambda \cdot t^*}} + \frac{\lambda \cdot t^* \cdot e^{-\lambda(T_C + t^* - 0.5)}}{(1 - e^{-\lambda \cdot t^*})^2} \right] \tag{6}$$

In order to examine the effects of different headway distributions on ramp merging capacity, the following steps were undertaken. Using eqn (6), possible merging capacity calculations were undertaken for both the negative exponential distribution and the revised new headway distribution proposed in this study. The results are shown in Figures 4 and 5 respectively. When using the negative exponential distribution, the possible merging capacity is independent of ramp volume levels. However, when the revised headway distribution is used, the possible merging capacity differs according to ramp merging volumes between 0 and 1,200 vehicles/hour. The changes in possible merging capacity with changes in the critical gap of the ramp merging vehicles should also be noted.

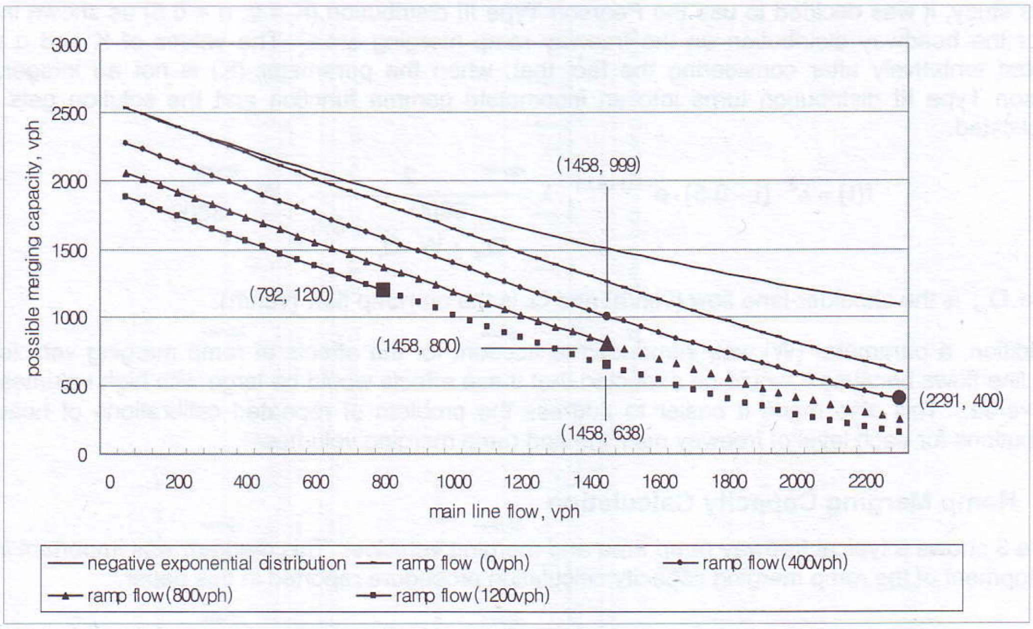

Figure 4: Possible merging capacity curves (Tc = 2 seconds)

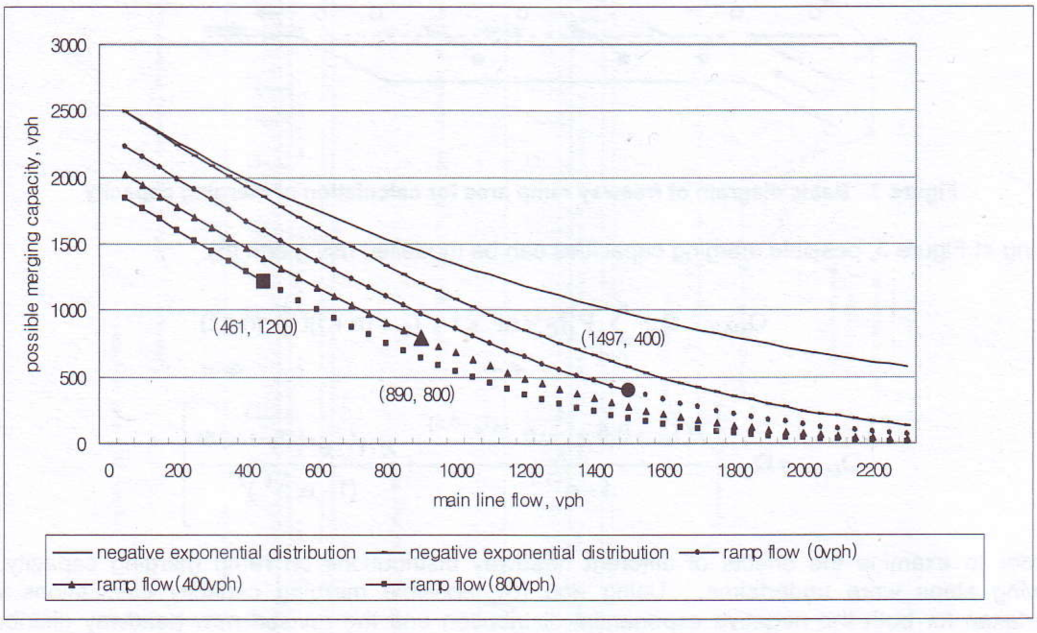

Figure 5: Possible merging capacity curves (Tc = 3 seconds)

In Figures 4 and 5, a set of points located right on the boundary line between the solid region and the dot region are clearly shown. This is the actual merging capacity or, alternatively, the number of vehicles entering the freeway ramp areas. The entering vehicles come from the freeway mainline as well as the ramp.

Figure 6 shows the final calculated merging capacity. In this Figure, 1) represents the situation when the freeway mainline flow is 792 veh/h and the ramp flow is 1,200 veh/h. If the freeway mainline flow increases to 1,458 veh/h, then the merging capacities decrease from 1,200 veh/h to 638 veh/h and the merging ramp flow becomes 638 veh/h. The possible merging capacity determined by the mainline flow (1,458 veh/h) and the merging ramp flow (638 veh/h) should therefore be 875 veh/h. By iterating the computational processes, the possible merging capacity converges to 800 veh/hour when the mainline volume is 1,458 veh/hour. The merging capacity curve shown in Figure 7 can be determined using the possible merging capacity curves shown in Figure 4.

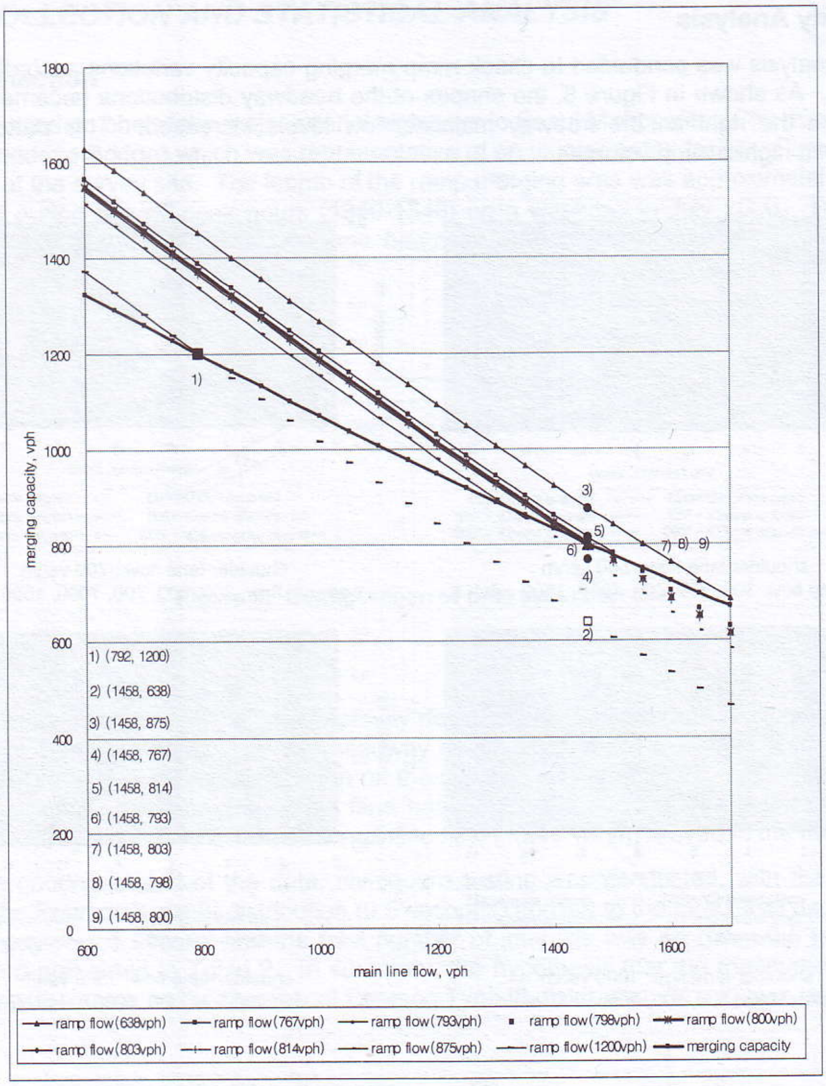

Figure 6: Example of variation in merging capacity (Tc = 2 seconds)

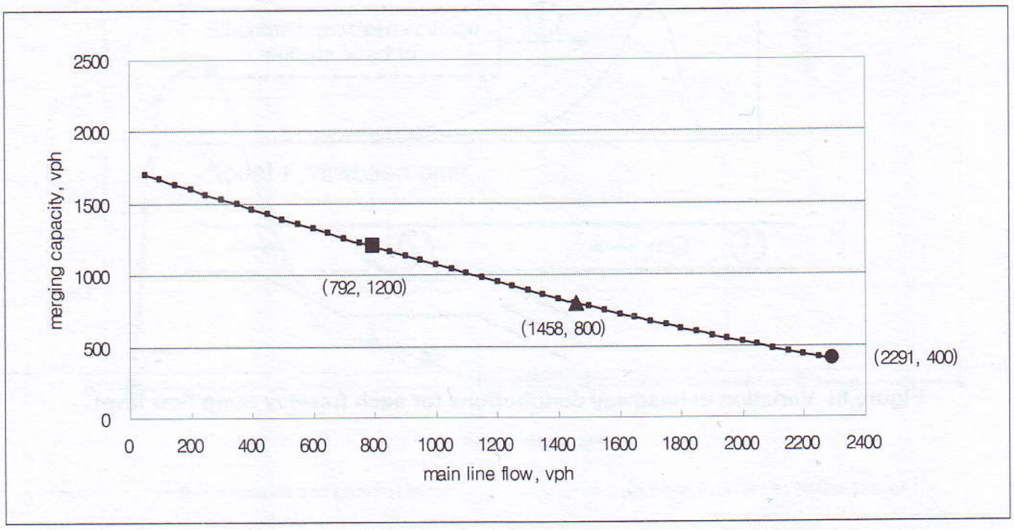

Figure 7: Merging capacity curve (Tc = 2 seconds)

3.3 Sensitivity Analysis

A sensitivity analysis was conducted to check ramp merging capacity variations according to different volume levels. As shown in Figure 8, the shapes of the headway distributions became more peaked and skewed to the right as the freeway mainline flow levels increased. This pattern was more pronounced with higher ramp volumes.

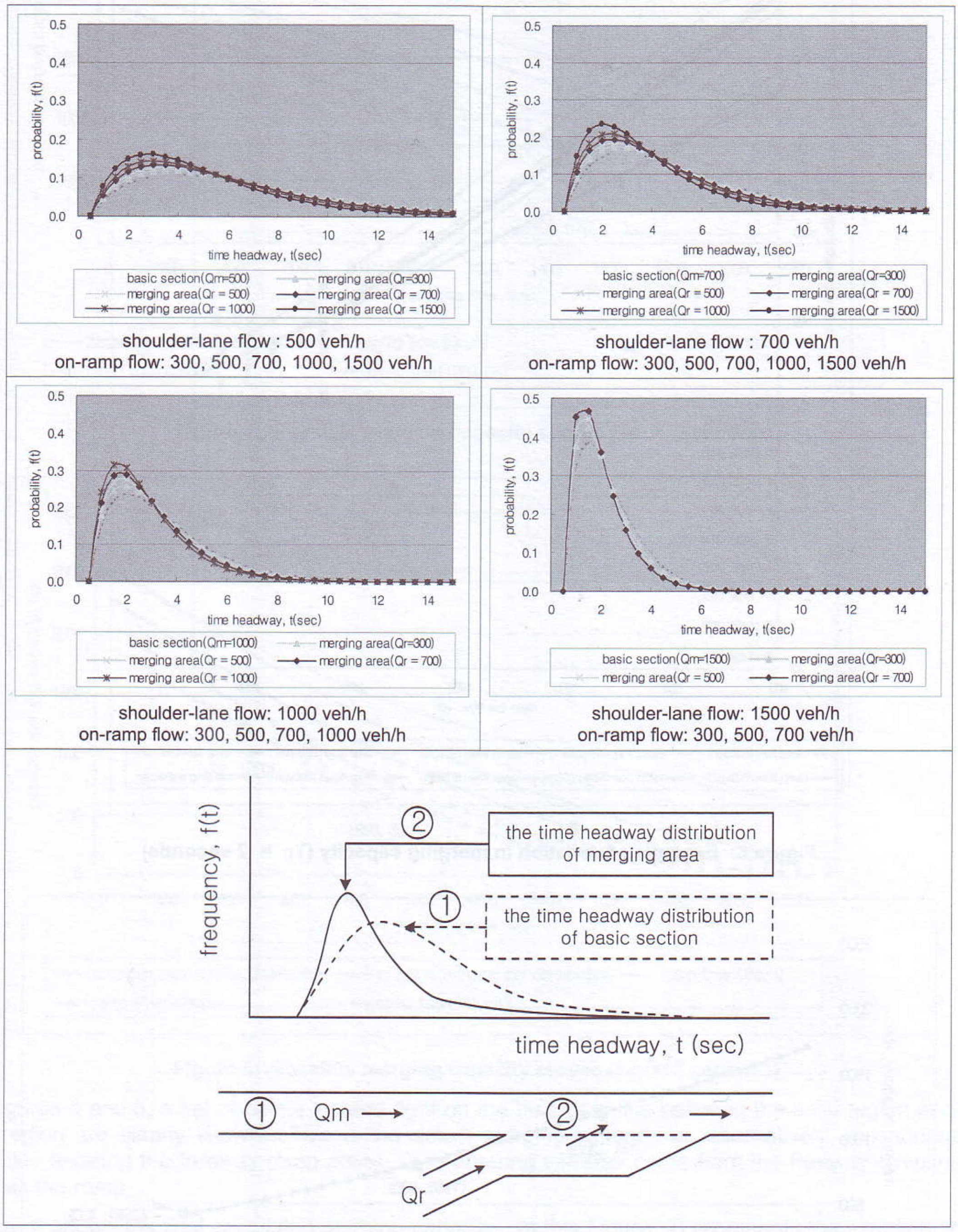

Figure 8: Variation in headway distributions for each freeway ramp flow level

4. DATA COLLECTION AND STATISTICAL ANALYSIS

4.1 Data Collection

Data was collected on one of the major arterial highways in Seoul Metropolitan City (Kangbyounbuk-ro near the Youngdong Bridge) which was representative of an uninterrupted facility. Figure 9 shows the configuration of the survey site. The length of the ramp merging area was approximately 200 m. Data was collected during two off-peak hours (1340-1540) on a weekday in July 2000. Traffic detectors were used to collect traffic volume and time headway data on the mainline and the ramp at the merging area.

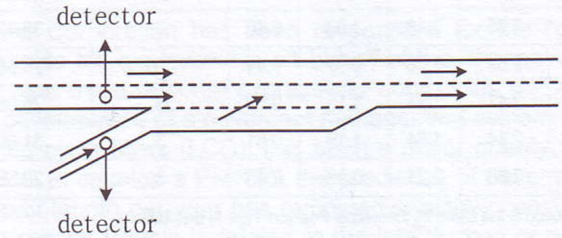

Figure 9: Configuration of data collection site

4.2 Results of Analysis

Figure 10 compares the theoretical time headway distribution (Pearson Type III distribution, K = 2) for the basic section and the measured time headway distribution on the merging area. It can be seen that the measured time headway distribution on the merging area was more peaked and showed less skewness to the right than the theoretical time headway distribution for the freeway basic section. There was no change in flow rate when the mainline flow (1,340 veh/h) arrived in the merging area.

To check the goodness-of-fit of the data, chi-square testing was conducted, with the minimum time headway of the Pearson Type III distribution (0.5 seconds) applied to the measured data. The interval of time headway was 1 second and the total number of intervals was six (from 0.5 to 6.5 seconds). The results are presented in Table 2. In summary, the hypothesis that the measured time headway distribution was the same as the theoretical Pearson Type III distribution (K = 2) was rejected.

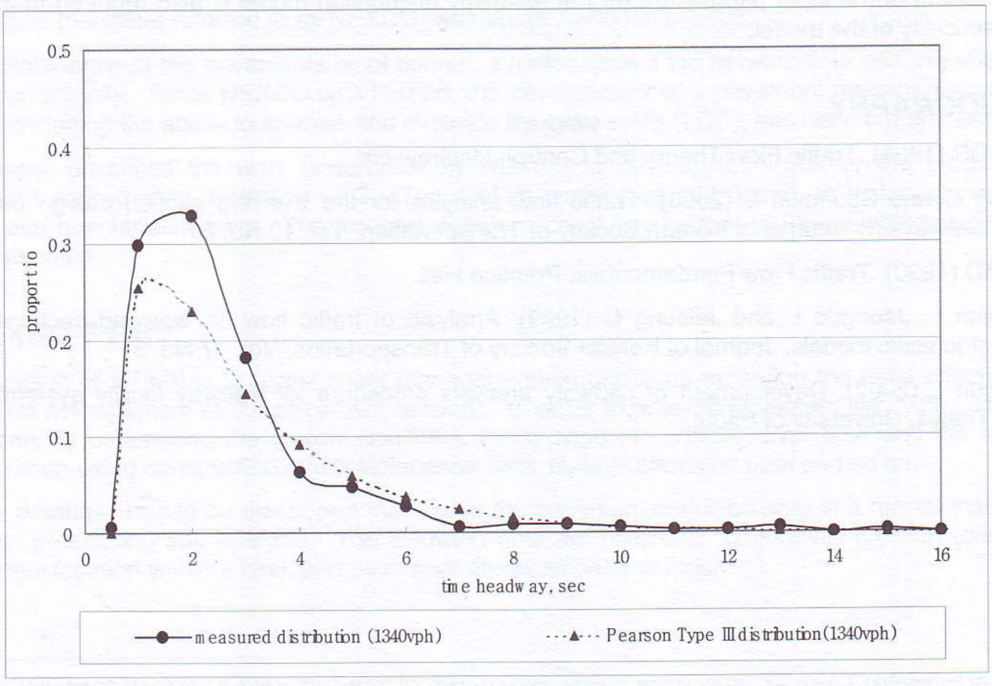

Figure 10: Comparison between theoretical Pearson type III distribution and measured time headway distribution

Table 2: Results of Chi-Square Testing

Time	Shoulder-lane flow (veh/h)	$\bar{t}$	s	Estimating parameter (p)		Degree of freedom = (l-1) - p	Chi-Square value	Chi-Square value	
								α = 0.05	α = 0.025
				$\bar{K}$	λ			7.82	9.35
1340-1355	1,460	2.61	1.93	1.09	0.52	(6-1)-2 = 3	22.30	rejected	rejected
1355-1410	1,396	2.59	1.87	1.12	0.53	3	29.50	rejected	rejected
1410-1425	1,260	2.86	2.22	1.07	0.45	3	25.21	rejected	rejected
1425-1440	1,308	2.75	2.18	1.03	0.46	3	39.42	rejected	rejected
1440-1455	1,340	2.69	2.26	0.97	0.44	3	25.58	rejected	rejected
1455-1510	1,448	2.47	1.71	1.15	0.59	3	28.08	rejected	rejected
1510-1525	1,476	2.44	1.64	1.19	0.61	3	31.69	rejected	rejected
1525-1540	1,344	2.68	2.21	0.98	0.45	3	28.55	rejected	rejected

(Null Hypothesis : Measured distribution is the same as theoretical Pearson Type III distribution).

5. FINDINGS AND CONCLUSIONS

This paper has described a study in which a literature review was undertaken to determine what was the most relevant headway distribution for calculating the merging capacity of a freeway ramp. During this review, it was noted that the negative exponential distribution is normally used for describing headway distribution. However, because of vehicle interactions at freeway ramp areas, it is suggested that a headway distribution different from the negative exponential distribution could be adopted because applying an inaccurate time headway distribution will result in an overestimation of merging capacity on freeway ramp sections.

A mathematical model for the calculation of merging capacity which considered both the freeway mainline and ramp flows was developed and described. A sensitivity analysis was conducted to examine the impacts of different volume levels on freeway ramp merging capacity.

Further research is required to develop a simpler method of analysis for various volume levels. Recalibration of the input parameters for the headway distribution model is also required to increase the practicality of the model.

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Life Cycle Analysis for Japanese Expressway Pavements*

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ABSTRACT

The Japan Highway Public Corporation has been responsible for the construction, operation and maintenance of the nationwide toll expressways on behalf of the Japanese Government since 1956. In 2005 it was privatised, and three regional expressway companies (NEXCO) were formed. Since NEXCO was formed, the development of a pavement management system (PMS), including the ability to analyse and evaluate life cycle costs (LCC), has been a major priority. This paper describes the work undertaken by NEXCO to develop a PMS for the prediction of pavement performance, including LCC. Since 1994, the repair length per year has remained relatively constant despite the increase in service length. The main reason for this is related to the introduction of polymer modified binders in the asphalt from this time. Ride quality is becoming an increasingly important issue on Japanese expressways and, for this reason, NEXCO has been investigating the applicability of the International Roughness Index (IRI) to assess the ride quality of its network. When the LCC models were applied to one section of the network, the durability of the porous asphalt pavement was about 1.2 times to 1.5 times higher than that of the dense-graded asphalt, suggesting that porous pavement is an excellent option for the rehabilitation of pavements about ten years of age or older. NEXCO has also been investigating the applicability of the International Roughness Index (IRI) to assess the ride quality of its network and to assist in the setting of limits for its safe and efficient operation.

1. INTRODUCTION

The Japan Highway Public Corporation has been responsible for the construction, operation and maintenance of the nationwide toll expressways on behalf of the Japanese Government since 1956. In 2005 it was privatised, and three regional expressway companies – East, Central and West Nippon Expressway Company Limited – were formed. The total length of expressway managed by the three companies (hereafter referred to as NEXCO) was about 7,500 km in 2005.

The maintenance of the pavements is, of course, a major issue if the network is to perform effectively and economically. Since NEXCO was formed, the development of a pavement management system (PMS), including the ability to analyse and evaluate life cycle costs (LCC), has been a major activity.

This paper describes the work undertaken by NEXCO to develop the PMS for the prediction of pavement performance, including LCC. The system was developed based on historical pavement profile and maintenance data. The forecast deterioration of one section of the network using LCC is also presented.

2. NEXCO'S PMS

The function of a PMS is to assist asset managers make decisions regarding the most effective and economic management of the pavement network. In order to analyse pavement life cycle costs, it is necessary to understand the current condition, the change in condition over time and the forecast deterioration using construction and maintenance data, surface condition data and so on.

First, a database has to be developed that stores the pavement condition data in a format that allows for easy processing and analysis. The following data are recorded: longitudinal location (distance), transverse location within a lane, and pavement composition (see Figure 1).

* This paper was awarded a Katahira Award for 'highly commended' paper at the REAAA Conference in Manila in November 2007.

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Horizontal coordinates include location within a lane in the transverse direction and every 1 meter in the longitudinal direction (see Figure 2). The development of a layer structure enables data to be extracted, based on the layer and plane coordinates (see Figure 3).

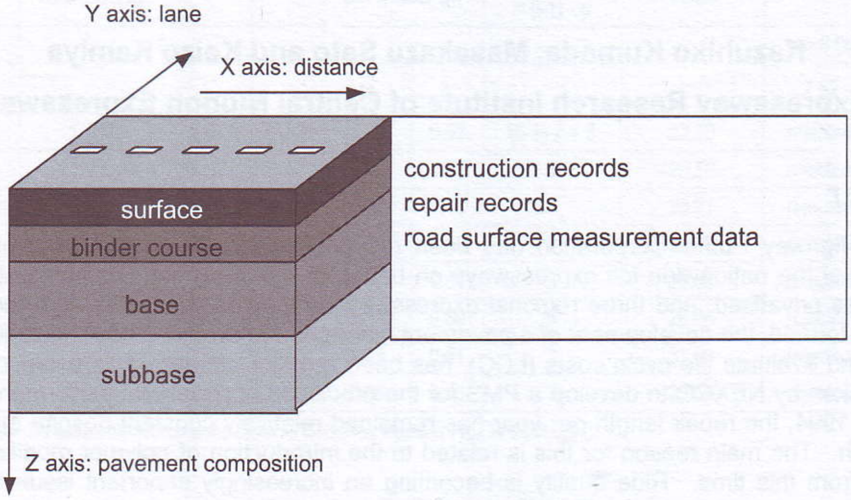

Figure 1: Pavement component of PMS

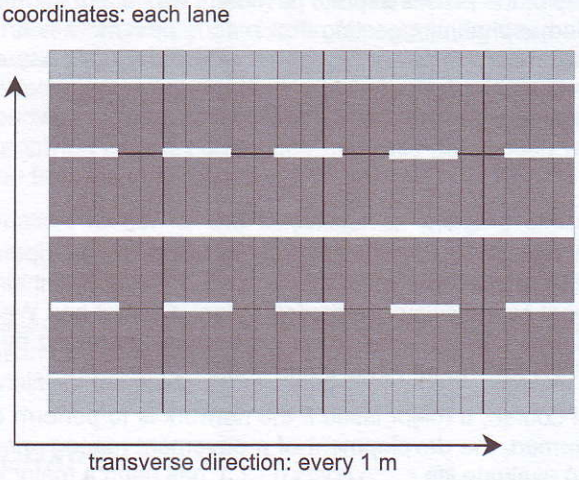

Figure 2: Plane coordinates of PMS

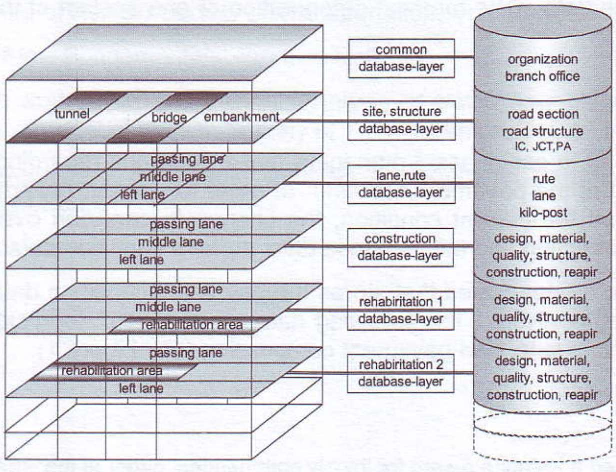

Figure 3: Layer image of PMS

3. ANALYSIS OF MAINTENANCE DATA

Figure 4 shows the relationship between service length of the NEXCO network and repair length between 1966 and 2000. It can be seen that the service length is increasing by about 200 km per year and that repair length is also increasing, especially between 1986 and 1993, when the repair length increased significantly. This reflected the fact that a large proportion of the network was nearing the end of its life during this period. In fact, about 70% of the network is over 10 years old.

However, since 1994, the repair length per year has remained relatively constant (2,000 lane-km per year) despite the increase in service length. The main reason for this is related to the introduction of polymer modified binders (PMBs) in the asphalt from this time. Asphalt composed of PMBs is much more durable than dense-graded asphalt.

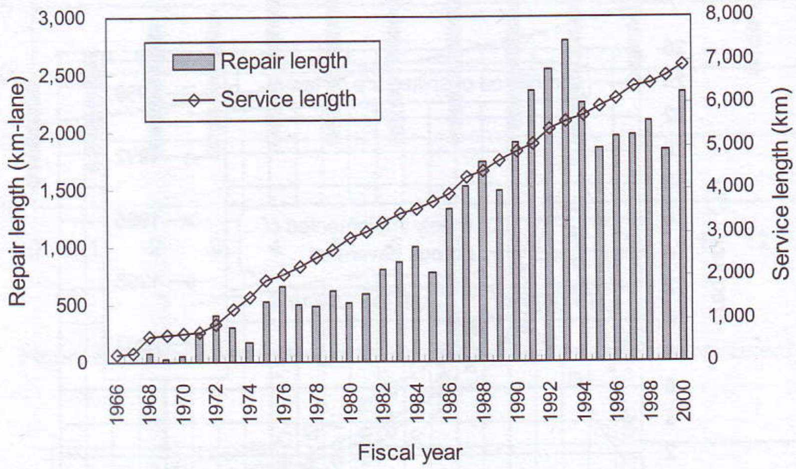

Figure 4: Relationship between service length and repair length

In addition, the type of pavement damage has also changed since PMBs were introduced. Figure 5 shows the type of damage each fiscal year. It can be seen that the main damage factor is rutting, which accounted for about 70% of all damage in 1994. Since 1994, however, the proportion of pavements cracking, compared to rutting, has been increasing. The increase in cracking is also related to the increase in low temperature cracking on lightly-trafficked sections of newly-opened pavements in cold regions, and the decrease in bearing capacity of pavements on heavily-trafficked sections which are reaching the end of their service life.

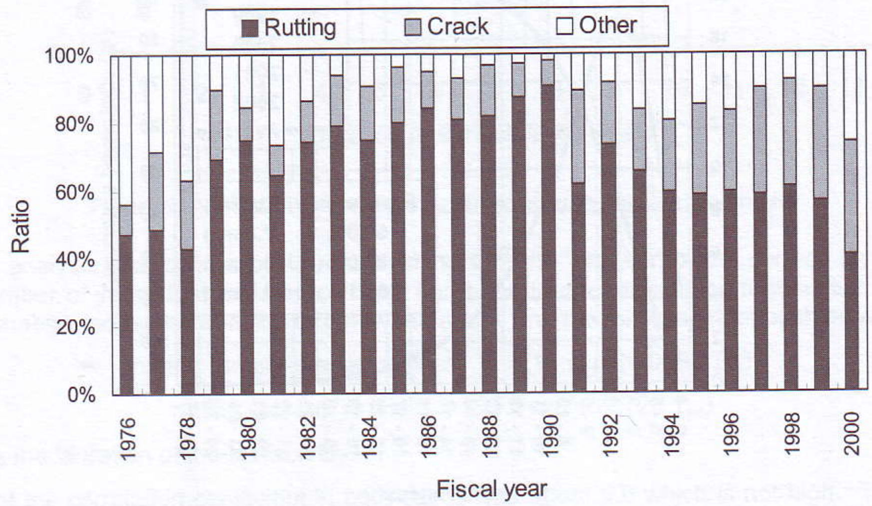

Figure 5: Causes of pavement damage (1976-2000)

4. ANALYSIS OF RUTTING DATA

As already discussed, rutting has been a major cause of distress to the Japanese expressway network. For this reason, the accumulation of rutting over time has been the main performance characteristic measured. About 170,000 lane-km of rutting data was collected between 1989 and 2002.

Figure 6 shows the frequency distribution of rutting by fiscal year. It can be seen that the frequency of rutting greater than 10 mm is decreasing, whilst rutting of 8 mm and less is increasing. This is related to the fact that the use of spiked tyres was prohibited by law in 1990 and the introduction of porous pavement (incorporating modified binders) in 1998. Figure 7 shows the situation in terms of rutting between 1998 and 2002. Over this period, the average rutting is about 7.3 mm and the frequency of rutting in excess of 20 mm is less than 1%.

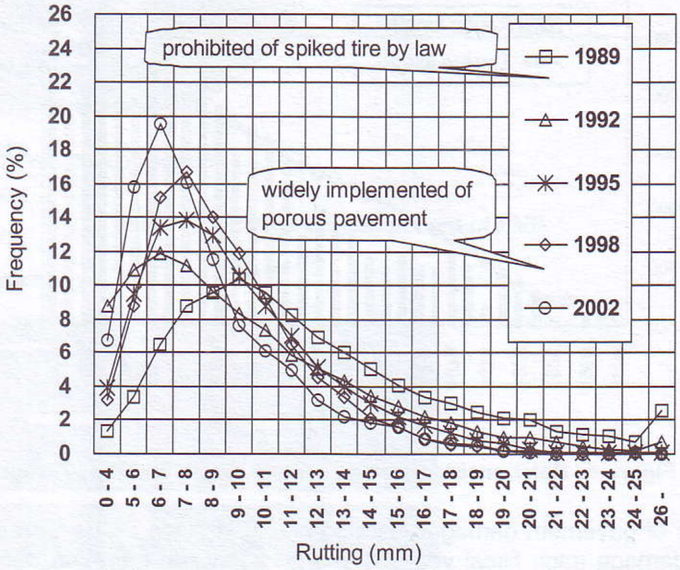

Figure 6: Frequency distribution of rutting by fiscal year

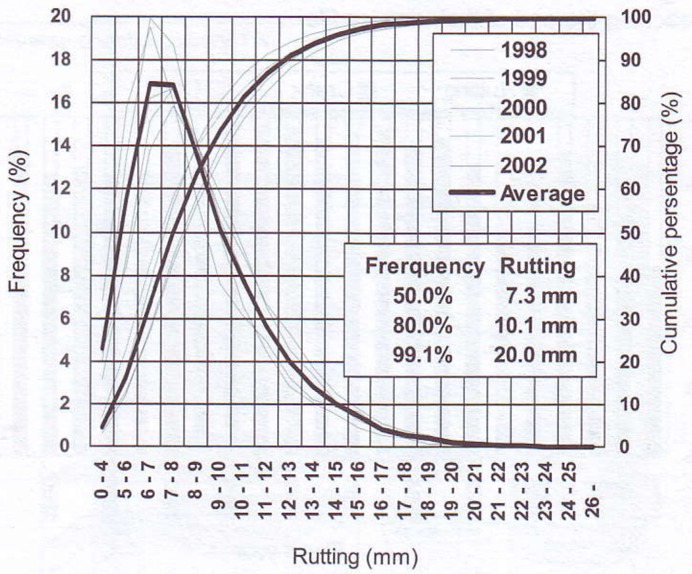

Figure 7: Current frequency of rutting

Figures 8 and 9 show the rate of increase in rutting of dense-graded asphalt pavements and porous asphalt pavements respectively on the Kanetu Expressway, a north-bound radial route from Tokyo. The rate of increase in rutting of the dense-graded asphalt is about 0.9 mm per year, compared with 0.6 mm per year for the porous asphalt.

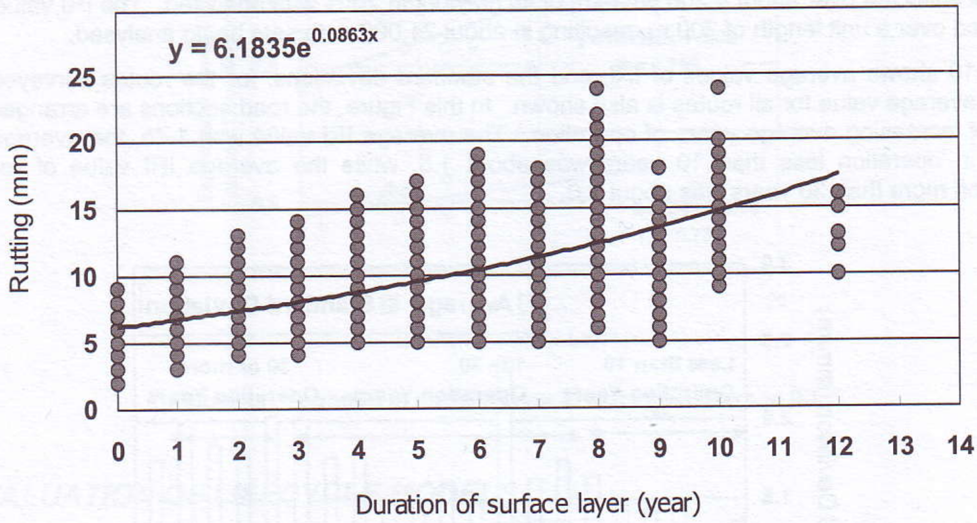

Figure 8: Variation in rutting over time: dense-graded asphalt pavement

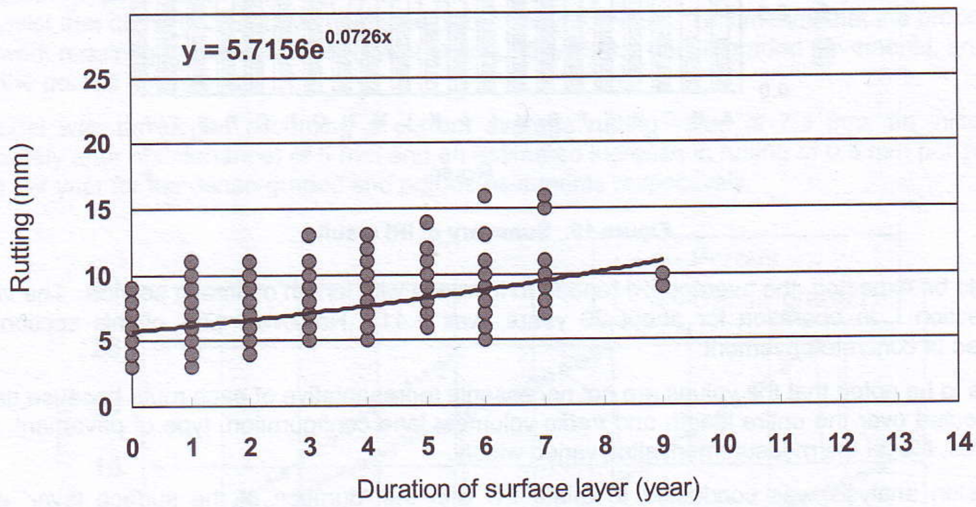

Figure 9: Variation in rutting over time: porous asphalt pavement

Regression analysis was conducted to relate rutting with the 'duration of the surface layer' – i.e. the average number of years that the surface layer had been in operation since that section of road had last been rehabilitated by the addition of a new material – and the following relationships were derived:

$$\text{rutting (dense-graded asphalt)} = 6.18 \exp\text{\textasciicircum}(0.0863\ T_{su}) \quad (1)$$

$$\text{rutting (porous asphalt)} = 5.72 \exp\text{\textasciicircum}(0.0726\ T_{su}) \quad (2)$$

where T_{su} is the 'duration of the surface layer'.

The value of the correlation coefficient in both cases was about 0.6 which is not high. This could be related to the fact that not all the maintenance data was available. Further research is therefore necessary.

5. ANALYSIS OF ROUGHNESS DATA

Ride quality is becoming an increasingly important issue on Japanese expressways. For this reason, NEXCO has been investigating the applicability of the International Roughness Index (IRI) to assess the ride quality of its network and to assist in the setting of limits for its safe and efficient operation.

IRI data collected over about 4,200 lane-km of its network in 2001 was analysed. The IRI values were averaged over a unit length of 200 m, resulting in about 21,000 data sets being analysed.

Figure 10 shows average values of IRI, and the standard deviations, for the routes surveyed. The overall average value for all routes is also shown. In this Figure, the road sections are arranged in the order of increasing average years of operation. The average IRI value was 1.75, the average IRI of routes in operation less than 10 years was about 1.5, while the average IRI value of routes in operation more than 30 years was about 2.0.

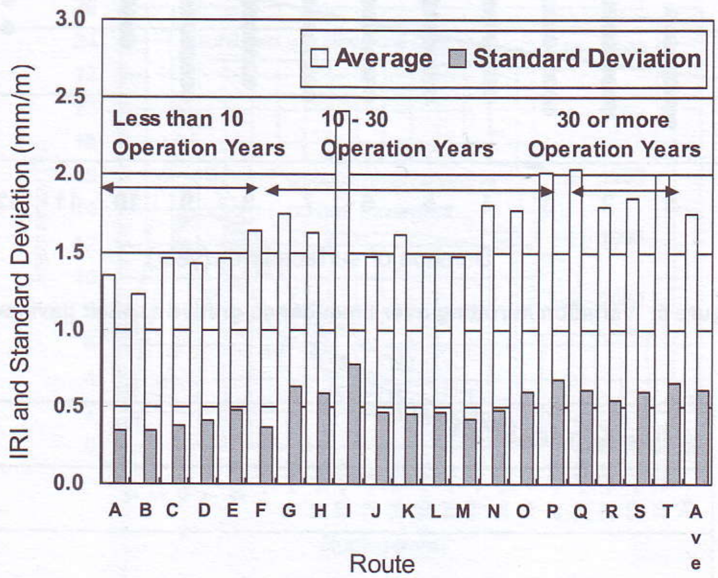

Figure 10: Summary of IRI results

As would be expected, the average IRI tended to increase with length of time in service. The value for road section I, in operation for about 20 years, was 2.41. However, 47% of this section length consisted of concrete pavement.

It needs to be noted that the values are not necessarily representative of each route because data was not collected over the entire length and traffic volumes, lane configuration, type of pavement, ratio of structures, etc. at the measurement sites varied widely.

Regression analysis was conducted to relate IRI with the 'duration of the surface layer' and the following relationships were derived.

$$\text{IRI (dense graded asphalt)} = 0.061T_{su} + 1.38 \quad (3)$$

$$\text{IRI (porous asphalt)} = 0.049T_{su} + 1.38 \quad (4)$$

where T_{su} is the 'duration of the surface layer'.

It can be seen from Figure 11 that the correlation coefficient in the case where 'years in operation' was used as the variable was 0.63 whereas when 'duration of surface layer' was used, the correlation coefficient was 0.77.

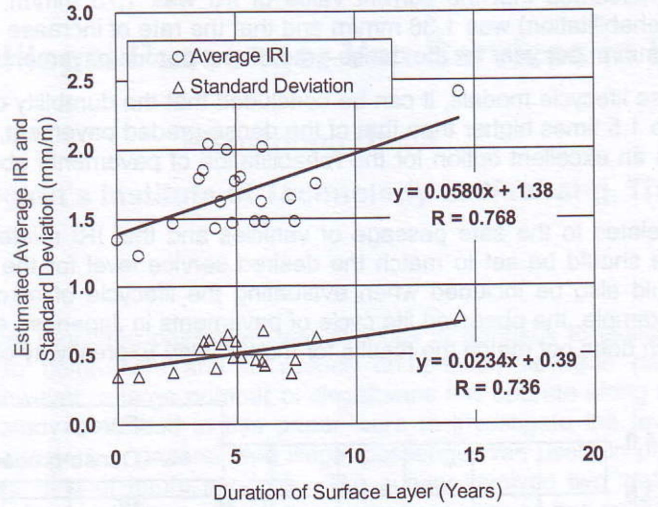

Figure 11: Duration of surface layer v. estimated average IRI

6. EVALUATION OF LIFECYCLE MODELS

The deterioration of a specific route within the network (Kanetsu) was estimated using the LCC models developed based on the result of the rutting and IRI analysis. Figure 12 shows the life cycle model for the dense-graded and porous pavements based on the rutting data. It can be seen that the rehabilitation cycle, assuming rutting of 25 mm, of the dense-graded pavement is approximately 20 years whilst that of the porous pavement is approximately 30 years. This means that the proportion of the network requiring rehabilitation each year is about 5% for the dense-graded pavements, and about 3% for the porous pavements.

The model was developed assuming a current average rutting value of 7.3 mm, an initial value (immediately after maintenance) of 5 mm and an estimated increase in rutting of 0.9 mm per year and 0.6 mm per year for the dense-graded and porous pavements respectively.

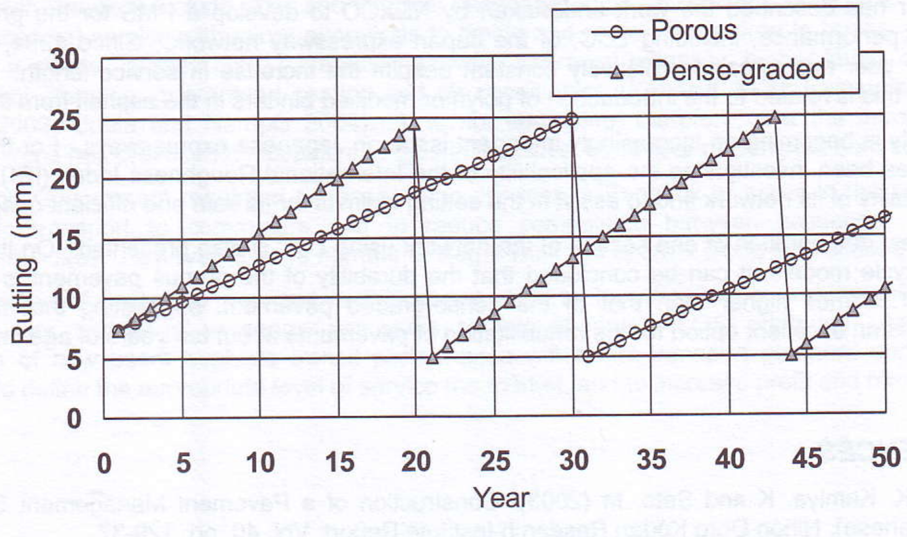

Figure 12: Lifecycle model (rutting)

Figure 13 shows the lifecycle model for the dense-graded and porous pavements based on the IRI data. It can be seen that, assuming that rehabilitation occurs when the rate of increase in IRI reaches 3.5 mm/m, the rehabilitation cycle for the dense-graded pavement is approximately 28 years whilst that for the porous pavement is approximately 35 years.

In this case it was assumed that the current value of IRI was 1.75 mm/m, the initial value of IRI (immediately after rehabilitation) was 1.38 mm/m and that the rate of increase in IRI was 0.061 mm/m per year and 0.049 mm/m per year for the dense-graded and porous pavements respectively.

On the basis of these lifecycle models, it can be concluded that the durability of the porous pavement is about 1.2 times to 1.5 times higher than that of the dense-graded pavement, demonstrating that the porous pavement is an excellent option for the rehabilitation of pavements about ten years of age or older.

Given that rutting relates to the safe passage of vehicles and that IRI relates to ride quality index, rehabilitation criteria should be set to match the desired service level for the road. However, other considerations should also be included when evaluating the lifecycle of a road including drainage, visibility, etc. For example, the observed life cycle of pavements in Japanese expressways is typically 10 to 15 years, which does not match the results for the Kanetsu Expressway presented in this paper.

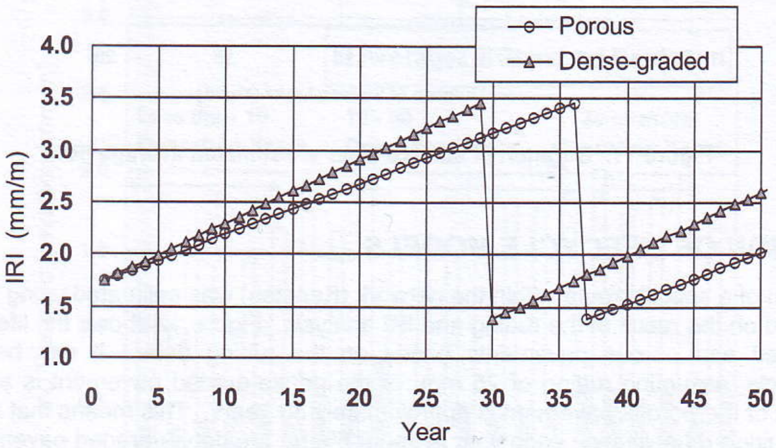

Figure 13: Lifecycle model based on IRI

7. CONCLUSIONS

This paper has described the work undertaken by NEXCO to develop a PMS for the prediction of pavement performance, including LCC, of the Japan expressway network. Since 1994, the repair length per year has remained relatively constant despite the increase in service length. The main reason for this is related to the introduction of polymer modified binders in the asphalt from this time.

Ride quality is becoming an increasingly important issue on Japanese expressways. For this reason, NEXCO has been investigating the applicability of the International Roughness Index (IRI) to assess the ride quality of its network and to assist in the setting of limits for its safe and efficient operation.

The forecast deterioration of one section of the network using LCC is also presented. On the basis of these lifecycle models, it can be concluded that the durability of the porous pavement is about 1.2 times to 1.5 times higher than that of the dense-graded pavement, suggesting that the porous pavement is an excellent option for the rehabilitation of pavements about ten years of age or older.

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The Use of Illegal Passenger Van Services in Bangkok*

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ABSTRACT

In 1999, the Government regulated passenger van services in Bangkok to assist in the provision of safe public transport to commuters and to reduce competition between passenger vans and conventional buses. However, a large number of illegal vans still operate along the licensed routes. The objectives of the study described in this paper were to investigate the level-of-service (LOS) attributes of public transit users in general, and illegal passenger van users in particular, and to use these findings in the planning of future services. The survey involved two stated preference (SP) experiments: a mode choice experiment considering passenger van and bus public transit modes, and mode choice between two alternative passenger van services. Both experiments considered different subsets of LOS attributes: walking distance, waiting time and comfort; and in-vehicle time, the number of stops and safety. In addition to the SP data, revealed preference (RP) data was also collected. Using these models, the willingness-to-pay for LOS improvements was derived, and future demand under alternative policy scenarios forecast. These findings will contribute to the future provision of illegal passenger vans in the transport system.

1. INTRODUCTION

Bangkok is the centre of industry, commerce, economy, education, administration and politics in Thailand. The area of the central area is 1,569 km^2 and the population is 6.3 million A total of 17.1 million person-trips occurs daily, of which 9.47 million take place on public transport systems (OCMLT 2001). Whilst the use of mass rapid transit is increasing, buses remain the most popular mode of public transport. Bus patronage is, however, in decline, due largely to severe traffic congestion and the resulting poor level of service. The growing popularity of para-transit can be attributed to its relative advantage over buses in terms of both service level and service area. The major modes of para-transit in Bangkok include taxis and informal passenger vans. Passenger vans offer a point-to-point (shuttle-like) service with minor alterations to timing and route. Compared to buses, passenger vans offer a shorter journey time, increased headway, improved reliability, higher levels of comfort, less frequent stopping, guaranteed seating and air conditioning (Kunasol 2000; Eamsupawat 2002; Nontasiri 2003; Upala and Narupiti 2004). It is not surprising, therefore, that the introduction of passenger vans has provoked noticeable changes in the travel behaviour of Bangkok commuters.

In 1999, the Government regulated passenger van services in Bangkok to assist in the provision of safe public transport to commuters and to reduce competition between passenger vans and conventional buses. However, a large number of illegal vans still operate along the licensed routes.

The objectives of the study reported in this paper were to investigate the preferences of public transport users, and illegal passenger van users, to level-of-service (LOS) attributes. An improved knowledge of how users evaluate transit performance will assist transport planners and transport agencies to define the appropriate level of service the market, and to increase profit and market share.

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2. LITERATURE REVIEW

2.1 Evaluation of User Preference

User preference evaluation is a significant tool in the evaluation of travel behaviour. One technique used to evaluate user preference is the 'stated preference' (SP) technique, which was developed in the early 1970s (McFadden 1973). Thesedays, user preference evaluation and travel behavioural research is generally conducted using a microeconomic approach or 'discrete choice' models. The discrete choice model, or 'random utility model' (RUM), initially relied on 'revealed preference' (RP) data, but SP data is now generally used, particularly for transport demand forecasting and valuation.

Using the SP technique, hypothetical questions are asked to discover how people respond to a range of choices. The economic value is then estimated in term of 'willingness to pay' (WTP) or 'willingness to accept' (WTA). Whilst the aim of the RP analysis is to deduce willingness to pay from observed evidence of how people behave when confronted with real choices (Pearce 2002), the SP technique is widely used in travel behavioural research for attributing valuations and assisting with demand modelling (Louviere et al. 2000; Ortuzar 2000; Bates 1998). The major steps include questionnaire design, data collection and data analysis. The experimental design is formulated using sets of hypothetical situations and payment scenarios, transport demand is forecast, and conclusions reached regarding willingness to pay for a service (Pearmain and Kroes 1990; Hensher 1994; Bateman et al. 2002).

2.2 Combining Revealed Preferen ce and Stated Preference Data

The combining of RP and SP data commenced with the work of Ben-Akiva and Morikawa (1991). The advantage of combining the data is that the two data types are complementary, that is, the strengths of one cover the weaknesses of the other. In particular, the credibility and realism of the RP data combines well with the efficiency and flexibility of the SP data. Bradley and Daly (1997) proposed the 'simultaneous estimation' approach which made use of a feature of ALOGIT software to postulate a single element nested logit model. Using this approach, a series of subsets of alternatives are defined in such a way that it is acceptable to apply the multinomial model within the subsets. Choice between the subsets is modelled either by using the multinomial model or by defining further subsets in a multi-level tree structure (HCG 2000). The tree logit model addresses the limitations of the probit form of the Ben-Akiva and Morikawa approach, which is not at present suitable for large-scale modelling under the tight time and cost constraints of practical studies, and the sequential approach of Swait and Louviere (1993).

2.3 Transit Level-Of-Service (LOS) and Discrete Choice Theory

Transit level-of-service (LOS) is a complex and dynamic process in which the LOS can change according to inconsistencies of circumstances or a person's thought processes. It is assumed that travellers will make decisions that are in their own self-interest. In other words, they will optimise a set of variables by assessing the LOS for the various options and select the most acceptable option (Sussman 2000). McFadden (1978) used an axiom of choice theory in which it is assumed that, that among the available alternatives, the one that maximises utility will be selected. This work is the basis of disaggregate demand forecasting models, and induced change in behavioural research and evaluation models.

3. METHODOLOGY

The mode choice models used in this study were based on the principle that a decision-maker will choose the mode that yields the greatest satisfaction or 'utility' of transit LOS. Utility is postulated to be a function of both observable (or deterministic) utility and unobservable (or random) utility:

$$U_{ni} = V_{ni} + \varepsilon_{ni} \tag{1}$$

where V_{ni} is the deterministic utility derived from alternative i by decision-maker n, and ε_{ni} is the associated random utility.

In the analysis, it was assumed that ε_{ni} was independent and identically Gumbell-distributed (IID assumption) and that the ratio of the choice probability for the decision-maker was unaffected by the systematic utilities of all other alternatives (independence from irrelevant alternatives, IIA, property). The multinomial logit model (MNL) was applied to model decision-maker behaviour relating probability of choice to utility as follows

$$P_{ni} = \frac{e^{\mu V_{ni}}}{\sum_{ni' \in J} e^{\mu V_{ni}}} \tag{2}$$

where P_{ni} represents the probability of the decision-maker n to select option i.

An important practical issue is the specification of V_{ni}, which is typically represented as a function of observed variables relating to the alternative. All the following models were based on the transit (LOS) attributes, with μ being a strictly positive scale parameter.

The models used to combine the RP and SP data were based on the tree logit assumption. The initial notation is similar to the approach reported by Morikawa (1994). The (latent) utility maximised by travellers in their revealed preference (U^{RP}), and the stated preference for a given traveller (U^{SP}), for a given alternative, is given by:

$$U^{RP} = \beta.X^{RP} + \alpha.Y + \phi \tag{3}$$

$$U^{SP} = \beta.X^{SP} + \gamma.Z + \varphi \tag{4}$$

where X^{RP}, Y, X^{SP} and Z are vectors of the measured variable influencing the RP, SP decision

β, α, γ are vectors of unknown parameters (to be estimated)

ϕ, φ represent the sum of the unmeasured components of the utility influencing the RP and SP decision.

The joint-estimation approach lies in the variables, X, that appear in eqns (3) and (4). The coefficient, β, can be estimated using the information from both the RP and SP surveys assuming that the mean value of the unmeasured components, ϕ, and φ is zero for each alternative.

Using the Gumbell or Weibull assumption, ψ^2 was defined as the ratio of the RP and SP variances, and then the SP utility can be scaled by ψ where:

$$\psi^2 = var(\phi)/var(\varphi) \tag{5}$$

$$\psi.U^{SP} = (\psi.\beta).X^{SP} + (\psi.\gamma).Z + (\psi.\varphi) \tag{6}$$

This equation can be used when the random variable in the new SP utility form has a variance equal to the RP utility form.

The mean utility of each of the 'dummy' alternatives is usually composed of a tree-logit model for SP observations:

$$V^{COMP} = \psi.\log(\sum \exp(V^{SP})) \tag{7}$$

In eqn (7), all the alternatives are summed in the nest corresponding to the composite alternative. It is simply the measured part of the SP utility. Then, because each 'nest' contains only one alternative in this specification:

$$V^{COMP} = \psi.U^{SP} = (\psi.\beta).X^{SP} + (\psi.\gamma).Z \tag{8}$$

The analytical framework is then easily created within the artificial tree-logit structure. The RP alternatives are placed just below the root of the tree, while the SP alternatives are placed in a single-alternative nest.

Unless otherwise specified, all the models are based on linear-in-parameters formulations of the utility functions which was adequate for the needs of this study.

This may be expressed as:

$$V_{ni} = \beta_{fare} \cdot X_{fareni} + \sum_k \beta_k \cdot X_{nik} \tag{9}$$

where X_{nik} are observations relating to the k^{th} variable (or 'attribute') of decision-maker n and alternative i, and the β parameters need to be estimated. X_{fareni} are observations relating to the fare attribute of decision-maker n and alternative i, and β_{fare} are associated fare parameters.

Under general conditions, any function can be estimated in the linear-in-parameters form. A further attraction of the linear-in-parameters functional form is that, by taking ratios of parameter estimates to the estimate of a fare parameter, the marginal rate of substitution with respect to fare (cost), or 'value', can be readily inferred. The form of marginal willingness to pay, or value for any one of the option attributes, may be written as:

$$\text{marginal willingness to pay (MWTP)} = \frac{\beta_k}{\beta_{fare}} \tag{10}$$

where β_{fare} is the fare coefficient and β_k is the level of service attribute k coefficient.

4. MODEL DEVELOPMENT

The principal survey tool was a questionnaire combining RP and SP data collection. The main survey was based on 300 illegal van users which were drawn from three strata pertaining to different spatial locations: inner-city, urban-fringe and suburban. The survey involved two basic SP experiments with seven attributes. The LOS attributes were passenger vans and buses, whilst the seven attributes identified and included in the SP design to represent the operational characteristics of the transit mode and its service were: fare, waiting time, walking distance, vehicle comfort, in-vehicle travel time, number of stops and driver behaviour (in terms of safety).

Initial modelling of the pilot data using elementary logit formulations involved both mode choice and within-mode choice. The results confirmed the significance of the LOS attributes, and confirmed the suitability of the chosen survey tool. The SP simulation was also used to adapt the choice experiments before the collection of data for the main survey (Fowkes 2000; Bradley and Daly 2000).

4.1 Structure of SP Survey

The survey involved two basic SP experiments: a within-mode choice between two alternative passenger van services; and a mode choice experiment considering the public transit modes of passenger vans and buses. Both experiments were further separated into two, each part containing a different subset of LOS attributes, but including the common attribute of fare to permit subsequent merger. The first part represented 'before passenger van usage' and focussed on walking distance, waiting time and vehicle comfort, whilst the second part represented 'after passenger van usage' and included in-vehicle time, number of stops and driver behaviour. In addition to the SP data, RP data was also collected from each respondent with the design of level of attributes related to the current passenger van and bus services. The questionnaire was divided into six main parts: personal characteristics and opinions, RP illegal van, within-mode choice SP game 1, within-mode choice SP game 2, between mode choice SP game 3, and between mode choice SP game 4.

5. RESULTS

User characteristics and opinions were analysed using SPSS for Windows (SPSS 2002) whilst the discrete choice modellings was estimated using the ALOGIT software (HCG 2000).

5.1 User Characteristics

The majority of the respondents were female (51.30%), the average age was 21 years, average income 5,296.05 Baht/month†, average household income 24,643 Baht/month, average car ownership 1.09 vehicles/household, and frequency of use of passenger van 4.30 times/week. The majority of the respondents (81.00%) were unaware that they were using an illegal van.

† 1000 THB ≈ US$31 (March 2007).

Table 1 presents the distribution access and egress mode to passenger van usage for the feeder-distributor mode. In order to access to the passenger van service, respondents were using conventional buses (40.7%), walking (27%), air-conditioned buses (21.7%) and motorcycle taxi (7.3%). Egress to the destination was walking (35.7%), conventional buses (30%), air-conditioned buses (20.7%) and motorcycle taxi (9.3%). Most respondents preferred to use the passenger van service because it was faster (61%), convenient (18.3%), comfortable (10%), reliable (6.7%), lower cost (2.3%), and safe (1.7%). The respondents rated the passenger van LOS at 4.29. This result represents a medium to high level ('1' being the lowest LOS and '7' the highest).

Table1: User Characteristics

Variable		Frequency	Percentage
Gender	male	142	47.3
	female	158	52.7
Access mode	walking	81	27
	conventional bus	122	40.7
	air-conditioned bus	65	21.7
	motorcycle taxi	22	7.3
	other modes	10	3.3
Egress mode	walking	107	35.7
	conventional bus	90	30
	air-conditioned bus	62	20.7
	motorcycle taxi	28	9.3
	other modes	13	4.3
Main reason for using passenger van	faster	183	61
	More reliable	20	6.7
	comfort	30	10
	convenience	55	18.3
	safety	5	1.7
	lower cost	7	2.3
Knowing using of van was illegal	yes	57	19
	no	243	81

Variable	Mean	Std Dev.
Age (years)	21.35	2.16
Income (Baht/month)	5,296.05	5,000.00
Household income (Baht/month)	24,643.42	20,000.00
Car ownership (vehicles/household)	1.09	1.00
Frequency of use (times/week)	4.30	2.99
LOS of passenger van	4.08	1.16

5.2 Valuation SP Model

Six of the seven attributes of the within-mode model (fare, walking distance, vehicle comfort, in-vehicle time, number of stops and driver behaviour) were significant at the 1% level, each with the expected sign. The results demonstrated an alternative-specific preference of 3.686 baht for passenger van service type-2 against passenger van service type-1. In terms of the generic attributes, walking distance was valued at 0.034 baht/metre, vehicle comfort 6.073 baht/level, in-vehicle time 0.189 baht/minute, number of stops 0.205 baht/point, and driver behaviour 8.452 baht/level.

For the between-mode model, five of the seven attributes (fare, waiting time, vehicle comfort, in-vehicle time and driver behaviour) were significant at the 1% level, each with the expected sign. Respondents demonstrated an alternative-specific preference of 10.208 baht for the bus service compared with the passenger van service. Of the generic attribute, waiting time was valued at 0.480 baht/minute, vehicle comfort 16.339 baht/level, in-vehicle time 0.398 baht/minute, and driver behaviour 19.561 baht/level.

5.3 Valuation RP-SP Model

For the analysis of RP-SP model, two models were considered: 2-choices (RP choice and SP choice) and 3-choices (RP choice and 2-SP choices). The 3-choice model was found to be superior to the 2-choice model for both the within-mode model and between-mode model.

For the within-mode model, six of the seven attributes (fare, walking distance, vehicle comfort, in-vehicle time, number of stops and driver behaviour) were significant at the 1% level, each with the expected sign. Respondents demonstrated an alternative-specific preference of 24.894 baht for the RP-van service and 4.509 baht for the passenger van service type-2 compared with the passenger van service type-1. Of the generic attributes, waiting time was valued at 0.136 baht/meter, walking distance 0.002 baht/meter, vehicle comfort 7.339 baht/level, in-vehicle time 0.088 baht/minute, number of stops 0.209 baht/point, and driver behaviour 7.820 baht/level.

For the between-mode model, respondents demonstrated an alternative-specific preference of 48.710 baht for the RP-van service and 11.121 baht the for bus service compared with the passenger service. Of the generic attributes, waiting time was valued at 0.052 baht/meter, walking distance was 0.003 baht/meter, vehicle comfort 17.438 baht/level, in-vehicle time 0.072 baht/minute, number of stops 0.238 baht/point, and driver behaviour 13.582 baht/level.

5.4 Comparison of Valuation SP Model with Jack-Knife Methods

To account for potential biases introduced by the repeat observation nature of the SP data, a jack-knife procedure was applied to the final model specification.

For the within-mode model, respondents demonstrated an alternative-specific preference of 3.681 baht for the passenger van service type-2 compared with the passenger van service type-1. Of the generic attributes, walking distance was valued at 0.036 baht/meter, vehicle comfort 6.073 baht/level, in-vehicle time 0.189 baht/minute, number of stops 0.205 baht/point, and driver behaviour 8.452 baht/level.

For the between-mode model, respondents demonstrated an alternative-specific preference of 9.667 baht for the bus service compared with the passenger van service. Of the generic attributes, waiting time was valued at 0.523 baht/minute, walking distance 0.038 baht/meter, vehicle comfort 14.755 baht/level, in-vehicle time 0.473 baht/minute, and driver behaviour 18.281 baht/level.

5.5 Comparison of RP-SP Model with Jack-Knife Methods

For the within-mode of the 3-choice model, respondents demonstrated an alternative-specific preference of 24.791 baht for the illegal RP van service and 4.492 baht for the passenger van service type-2 compared with passenger van service type-1. Of the generic attributes, waiting time was valued at 0.132 baht/meter, walking distance 0.002 baht/meter, vehicle comfort 7.348 baht/level, in-vehicle time 0.087 baht/minute, number of stops 0.104 baht/point, and driver behaviour 7.803 baht/level.

For the between-mode of the 3-choice model, respondents demonstrated an alternative-specific preference of 49.082 baht for the illegal RP van service and 11.281 baht for the bus service compared with the passenger van service. Of the generic attributes, waiting time was valued at 0.062 baht/meter, walking distance 0.005 baht/meter, vehicle comfort 17.579 baht/level, in-vehicle time 0.272 baht/minute, number of stops 0.238 baht/point, and driver behaviour 13.603 baht/level.

5.6 Valuation Merging SP Model

For the analysis of the merging SP model, the exploitation of merge SP models included four models: MNL, NL-1, NL-2, NL-3 (see Figure 1). When the four models were compared, the NL-3 model was found to be superior to the other models.

Respondents demonstrated an alternative-specific preference of 20.443 baht for the passenger van service, 11.833 baht for the passenger service type-1 and 6.926 baht for the passenger service type-2 compared with the bus service. Of the generic attributes, waiting time was valued at 0.126 baht/minute, walking distance 0.077 baht/meter, vehicle comfort 14.864 baht/level, in-vehicle time 0.193 baht/minute, number of stops 0.385 baht/point and driver behaviour 18.281 baht/level.

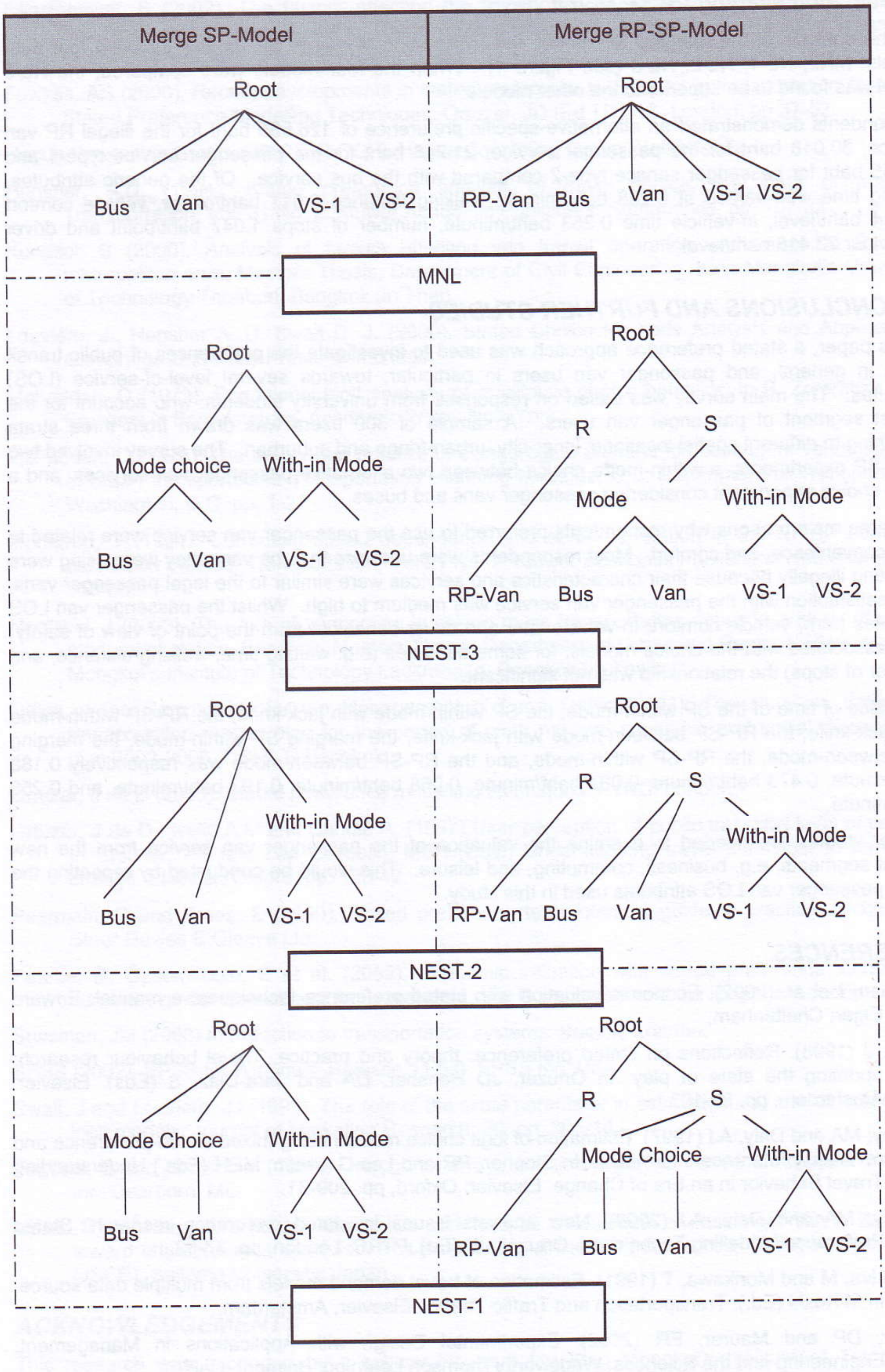

Figure 1. Artificial tree structure for merging SP and RP-SP

5.7 Valuation Merging RP-SP Model

For the analysis of the merging SP model, the exploitation of merge SP models included four sub-models: MNL, NL-1, NL-2, NL-3 (see Figure 1). When the four models were compared, the NL-3 model was found to be superior to the other models.

Respondents demonstrated an alternative-specific preference of 126.655 baht for the illegal RP van service, 30.018 baht for the passenger service, 21.248 baht for the passenger service type-1 and 17.555 baht for passenger service type-2 compared with the bus service. Of the generic attributes, waiting time was valued at 0.288 baht/minute, walking distance 0.011 baht/meter, vehicle comfort 24.014 baht/level, in-vehicle time 0.253 baht/minute, number of stops 1.047 baht/point and driver behaviour 23.418 baht/level.

6. CONCLUSIONS AND FURTHER STUDIES

In this paper, a stated preference approach was used to investigate the preferences of public transit users in general, and passenger van users in particular, towards several level-of-service (LOS) attributes. The main survey was based on responses from university students, who account for the largest segment of passenger van users. A sample of 300 users was drawn from three strata pertaining to different spatial locations: inner-city, urban-fringe and suburban. The survey involved two basic SP experiments: a within-mode choice between two alternative passenger van services, and a mode choice experiment considering passenger vans and buses.

The three main reasons why respondents preferred to use the passenger van service were related to time, convenience, and comfort. Most respondents were unaware that the vans they were using were operating illegally because their characteristics and services were similar to the legal passenger vans. User satisfaction with the passenger van service was medium to high. Whilst the passenger van LOS attributes (fare, vehicle comfort, in-vehicle time and driver behaviour from the point of view of safety) were associated with the choice models, for some attributes (e.g. waiting time, walking distance, and number of stops) the relationship was not significant.

The value of time of the SP within-mode, the SP within-mode with jack-knife, the RP-SP within-mode with jack-knife, the RP-SP between-mode with jack-knife, the merging SP within-mode, the merging SP between-mode, the RP-SP within-mode, and the RP-SP between-mode was respectively 0.188 baht/minute, 0.473 baht/minute, 0.087 baht/minute, 0.068 baht/minute, 0.193 baht/minute, and 0.253 baht/minute.

Further studies are needed to examine the valuation of the passenger van service from the new market segments, e.g. business, commuting, and leisure. This would be conducted by repeating the set of passenger van LOS attributes used in this study.

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Axle Load Equivalencies and the Effect of Wide Single and Dual Tyres on the Performance of Granular Pavements*

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ABSTRACT

ARRB Research is currently undertaking research to improve knowledge of the effects of axle load and tyre type on pavement deterioration. Two specific objectives of the research were to determine load damage exponents (LDEs) and to assess the relative damaging effects of wide single tyres and steer axle tyres compared to dual tyres for Australasian pavement types. This paper summarises the experimental program, analysis and findings of the assessment of the impacts of axle mass increases and tyre types on road deterioration for sprayed seal surfaced unbound granular pavements, the most common pavement type in Australasia. LDEs for deformation and roughness progression developed from the performance data collected under accelerated loading provided further evidence to support on-going use of the 4^{th} power LDE for granular pavements. It was also found that the relative wear (i.e. the number of cycles to achieve a permanent deformation of 20 mm) was 1.6 times, 3.8 times and 4.7 times for the 445/65R22.5 and 385/65R22.5 wide single tyres and 295/80R22.5 steer axle tyre respectively compared to the conventional dual tyres when all testing was conducted with the 40 kN half-axle ALF load. The reference load on a steer axle with 295/80R22.5 single tyres which was estimated to cause the same pavement wear (vertical surface deformation) as an 8.2 tonne Standard Axle fitted with dual 11R22.5 tyres was found to be 5.4 tonne, which is in good agreement with the recommendations contained in the Austroads Pavement Design Guide.

1. INTRODUCTION

ARRB Research is currently undertaking research to improve knowledge of the effects of axle load and tyre type on pavement deterioration. Two specific objectives of the research are to determine load damage exponents (LDEs) and to assess the relative damaging effects of wide single tyres and steer axle tyres compared to dual tyres for Australasian pavement types. This paper summarises the experimental program, analysis and findings of the assessment of the impacts of axle mass increases and tyre types on road deterioration for sprayed seal surfaced unbound granular pavements, the most common pavement type in Australasia. Full details of the work associated with the axle load equivalency testing are contained in Yeo, Martin and Koh (2006) whilst preparation of the final report on the work associated with the relative damaging effects of wide single and dual tyres is currently nearing completion.

2. LOAD DAMAGE RELATIONSHIP FOR UNBOUND GRANULAR PAVEMENTS

For unbound granular pavements, Australian practice is to define a 'Standard Axle' load as a single axle with dual tyres loaded to 8.2 tonne or 80 kN (Austroads 2004). Where the axle load differs from the Standard Axle load, the number of Standard Axle Repetitions (SARs) for the same damage may be calculated using eqn (1):

$$\text{Number of SARs for Same Damage} = \left[\frac{\text{Axle Load}}{\text{Standard Load for Axle Group Type}}\right]^{\text{LDE}} \quad (1)$$

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where SARs = Standard Axle Repetitions

LDE = Load Damage Exponent.

The commonly applied LDE value of 4 was originally derived from the AASHO road test data (Highway Research Board 1962). It represents an average exponent for a number of test pavements. This LDE was based on the number of applications of a Standard Axle load to cause the same reduction in Pavement Serviceability Index (PSI) as a given number of applications of a heavier load. The PSI represents a number of pavement attributes including measures of ride quality, rutting, cracking and asphalt patching. The aim of the research described in this paper was to generate performance data to enable an assessment of an appropriate LDE for unbound granular pavements typically used in Australasia.

3. ACCELERATED LOADING FACILITY

The Accelerated Loading Facility (ALF) is a mobile full scale, road test system which enables pavement performance to be assessed within a short time scale. A schematic diagram of ALF is shown in Figure 1, whilst the ALF specification is given in Table 1. Rolling wheel loads, which can be varied in 10 kN increments from 40 kN to 80 kN, are applied in one direction to pavement test strips 12 m long at a constant speed of 20 km/h. The load assembly, which tracks linearly, is guided by the main frame. The wheel is lifted off the pavement at the end of each cycle and supported by the main frame on its return.

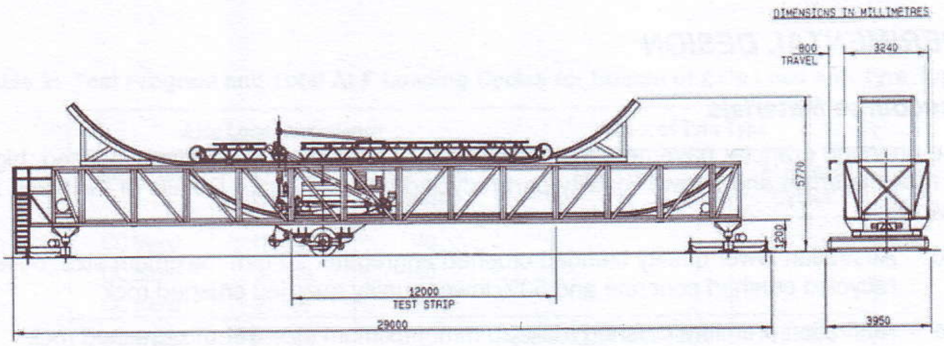


Figure 1: Schematic diagram of Australian Accelerated Loading Facility

Table 1: ALF Specification

test wheels	dual tyres (eg. 315/80 R22.2) or wide super single tyre (eg. 385/65 R22.5)
mass of test wheel assembly	40 kN to 80 kN in 10 kN steps
suspension for variable mass	air bag and shock absorbers
power drive to wheel	2 x 11 kW electric geared motors, uni-directional operation, wheels off pavement on return
transverse movement of test wheels	user programmable; typically a Normal distribution about 0.9 m or 1.2 m wide between outer edges of the dual tyres is used
test speed	nominally 20 km/h
cycle time	approximately 9.5 seconds
pavement test length	nominally 12 metres
site constraints	max. grade: 1%; max. crossfall: 3%
operation	automatic control system and fail-safe operation
portability	readily detachable and transportable
overall length	26.3 metres
overall width	4.0 metres (operating); 3.2 metres (transport)
overall height	5.7 metres (operating); 4.4 metres (transport)
total mass	approximately 45 tonne

Figure 2: Australian ALF operating indoors

For this project the ALF was operated at the ARRB indoor test facility, a shed 53 m in length and 18 m wide, located in Dandenong, Melbourne, Australia (Figure 2). The indoor environment enabled the ALF trafficking to be conducted under consistent dry moisture conditions which ensured the experiments at different axle loads or with different tyres were comparable.

4. EXPERIMENTAL DESIGN

4.1 Basecourse Materials

The three unbound granular pavement materials tested represented two commonly used, high quality crushed rock materials and a lower quality part recycled crushed rock. Details of the three materials are as follows:

CC Blend	Australian lower quality blended crushed aggregate: 20 mm maximum size, 50% blended recycled crushed concrete and 50% lower quality quarried crushed rock
Montrose	Australian premium crushed rock: 20 mm maximum size wet-mix crushed rock
NZM4	New Zealand premium aggregate: 40 mm maximum sized crushed alluvial gravel.

4.2 Axle Loads and Tyre Types

The experiment design included the current Standard Axle load (80 kN) adopted in Australia, represented by a half-axle load of 40 kN on the ALF as a control. Two increased half-axle loads of 60 kN and 80 kN were included for assessment of the effects of axle load on pavement wear.

For the assessment of the effect of tyre type and configuration on pavement wear, a series of experiments was conducted using a half-axle load of 40 kN. The tyres tested included the reference 315/80R22.5 dual tyres, two wide single tyres (445/65R22.5 and 385/65R22.5) and a steer axle tyre (295/80R22.5). These experiments were all conducted on the Montrose crushed rock pavement at a fixed tyre inflation pressure of 760 kPa (cold) for all tyres.

4.3 Pavement Structure

Three separate test pavements, each comprising a 200 mm thick layer of the basecourse material placed over a uniform sand subgrade, were constructed. All three test pavements were surfaced with a double sprayed bituminous seal. Figure 3 shows the typical pavement cross-section.

4.4 ALF Trafficking Program

A total of 11 ALF experiments were conducted, nine representing the original experimental design. Two additional experiments were conducted on the Montrose test pavement at 40 kN and 60 kN loading to improve the robustness of the data. Details of the experiments and the number of ALF cycles applied during each experiment are presented in Table 2. All the ALF axle load equivalency experiments were conducted using dual 315/80R22.5 tyres. Details of the experiments conducted to assess the effect of tyre type and configuration on pavement wear are also given in Table 2.

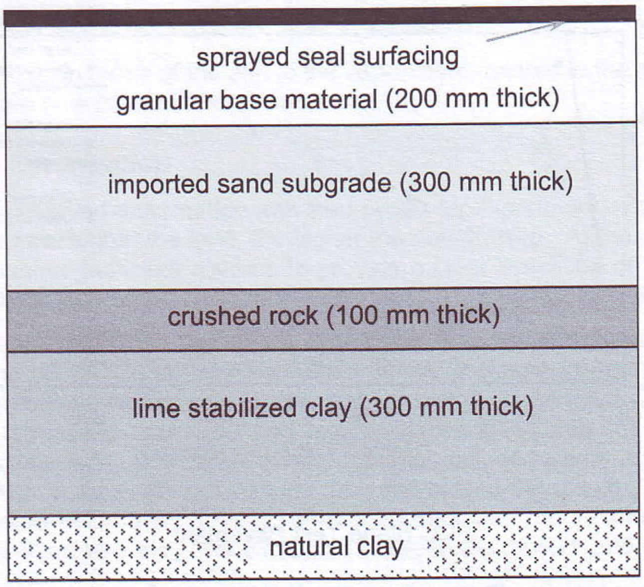


Figure 3: Typical test pavement cross section

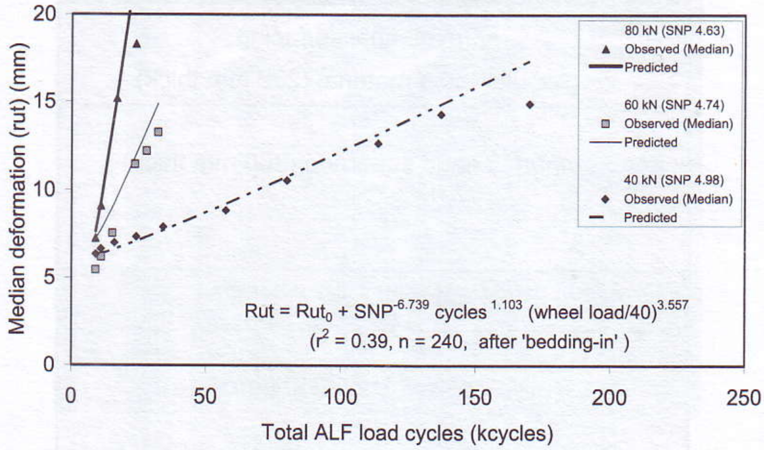
Table 2: Test Program and Total ALF Loading Cycles for Effects of Axle Load and Tyre Type

Axle Load Equivalency			**Effect of Tyre Type**	
Materials	**ALF Load Cycles**	**ALF Load (kN)**	**Tyre Type**	**ALF Load Cycles**
CC Blend	170,000	40		
CC Blend	32,000	60		
CC Blend	23,800	80		
Montrose	151,000	40	385/60R22.5 wide single	151,600
Montrose	169,000	40	445/65R22.5 wide single	152,400
Montrose	93,600	60	295/80R22.5 single steer	156,000
Montrose	90,000	60		
Montrose	50,000	80		
NZ M4	218,400	40		
NZ M4	111,600	60		
NZ M4	50,500	80		
Total	**1,159,900**			**460,000**

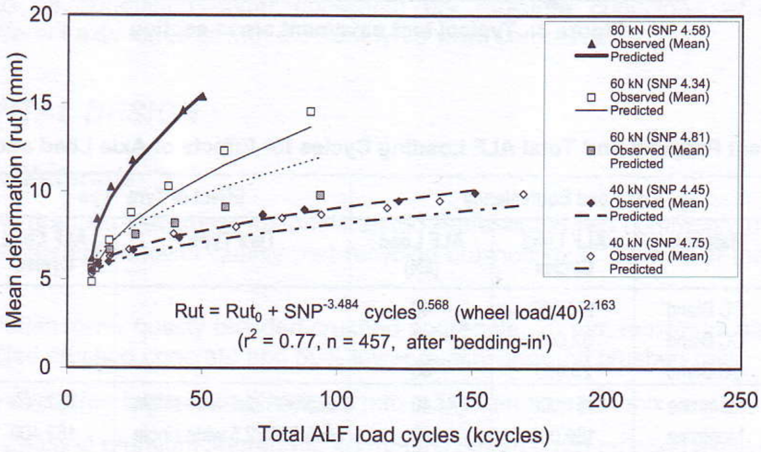
* All experiments conducted at 40 kN half-axle load

4.5 Performance Assessment

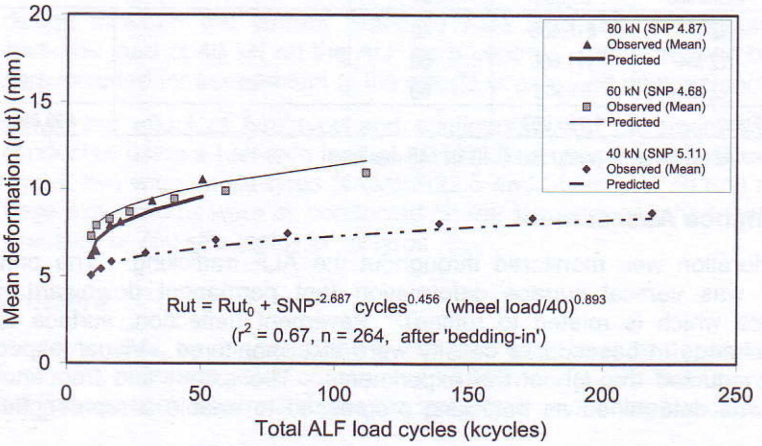
Pavement deterioration was monitored throughout the ALF trafficking. The primary indication of pavement wear was vertical surface deformation (net permanent downward movement of the pavement surface which is related to rutting). Pavement deflection, surface texture, change in roughness and change in basecourse density were also monitored. Visual inspection of pavement condition was conducted throughout the experiments. The extent and frequency of performance measurements was determined as trafficking progressed to enable a representative dataset to be collected.



CC Blend Test Pavement



Montrose Test Pavement



NZM4 Test Pavement

Figure 4: Observed deformation for each test pavement

5. PAVEMENT WEAR AT DIFFERENT AXLE LOADS

The most significant results, in terms of the aim of the experiment, related to the surface deformation data and these results are now presented and discussed.

5.1 Vertical Surface Deformation

Figure 4 compares the observed deformation with load cycles for experiment. The deformation results were as expected in that the higher the load, the higher the deformation. At the completion of loading, the test pavements exhibited sufficient distress to provide a clear indication of the relative effects of axle loads on performance.

For the CC Blend, the deformation in the 60 kN experiment was significantly higher than that in the 40 kN experiment. The 80 kN experiment showed a similar increase in deformation to the 60 kN experiment even though this pavement was slightly weaker based on the initial SNP values. The very similar and severe deformations under the two high loads indicated that the load capacity of the pavement had been exceeded. This lower quality material included some areas which were very weak and the deformation in these areas was very high and biased the results. Hence the data used in Figure 4 are the median deformation results rather than the mean deformation. For the other two, higher-quality, materials there was very little difference between the median and mean results.

For the Montrose crushed rock, deformation in the initial 60 kN experiment was only slightly higher than that in the 40 kN experiment. However, the initial 60 kN experiment was located on a stronger pavement than the 40 kN experiment (SNP of 4.20 cf. 4.11) which explains why the increase in load did not result in a significantly higher deformation. For the repeat 60 kN experiment the pavement was weaker than the 40 kN experiment (SNP of 4.04 cf. 4.11) and the rate of deterioration in this weaker test section was greater than that in the initial 60 kN experiment. The variations in SNP between experiments were taken into account in the determination of LDEs for each material.

5.2 Estimation of Load Damage Exponents

As discussed earlier, the vertical surface deformation data was used to derive the LDE values. The PSI-type approach used for the original derivation based on data from Highway Research Board (1962), could not be used as the observed changes in roughness were minimal for the two high-quality materials, there was minimal cracking and no asphalt patching was required.

A general relationship for vertical surface deformation (Rut) was formulated as follows:

$$Rut = R_0 + \Sigma\, \Delta rut \qquad (2)$$

where Rut = vertical surface deformation (mm)

R_0 = initial vertical deformation at 9000 cycles (mm)

$\Sigma\, \Delta Rut$ = cumulative rut progression after 9000 cycles of bedding in.

For each pavement material the progression of deformation after bedding in (ΔRut) was represented as a power function of SNP and standard axle repetitions (SARs) as follows:

$$\Sigma\, \Delta Rut = SNP^a \times SARs^b \qquad (3)$$

where SNP = adjusted structural number

SARs = $[\text{Wheel Load}/40]^{LDE} \times$ cycles

(based on eqn (1) for a single half axle dual tyre configuration)

Wheel Load = half axle dual wheel load (kN)

$$\text{LDE (Load Damage Exponent)} = c/b \qquad (4)$$

cycles = number of accelerated load cycles under a given wheel load after the initial 9,000 cycles of 'bedding in'

a, b and c = power coefficients (for gradual deformation phase).

Substituting for SARs and $\Sigma\, \Delta Rut$ in eqn (2) resulted in the following explicit relationships for deformation:

$$\text{Rut} = R_0 + \text{SNP}^a \times \text{cycles}^b \times (\text{Wheel Load} / 40)^c \quad (5)$$

using $$\Sigma\,\Delta\text{Rut} = \text{SNP}^a \times \text{cycles}^b \times (\text{ Wheel Load} / 40)^c \quad (6)$$

Eqn (4) was used to derive LDEs for pavements of a particular material over a range of wheel loads, based on the non-linear curve fitting of eqn (6) to the observed deformation data obtained from the ALF experiments.

Table 3 presents the model coefficients for eqn (6). These models are also shown in Figure 4, together with the observed deformation data. The LDE values were 3.2 for the CC Blend material, 3.8 for the Montrose material and 2.0 for the NZ M4 material. The range of LDEs found indicated that one LDE may not be appropriate for all unbound granular materials.

As with all accelerated pavement testing, the limitations of the experiments and analysis should be noted. The scope of the experimental work included three different test pavements but only one moisture environment was considered for each pavement. The sensitivity of the pavements to moisture ingress was not examined in these experiments. While limited in scope, this research has provided increased confidence and knowledge in the area of increased axle loading effects on pavement performance for unbound granular pavements.

Table 3: Estimated Model Coefficients and Load Damage Exponents (LDE) Based on Deformation

Test Pavement	Wheel Load (kN)	SNP	R (mm)	Model Coefficients# a	b	c	R^2	n	LDE
CC Blend	40	4.98	6.3	–6.739	1.103	3.557	0.39	240	3.22
	60	4.76	7.2						
	80	4.63	7.7						
Montrose	40	4.45	5.5	–3.484	0.568	2.163	0.77	457	3.81
	40	4.75	5.3						
	60	4.81	5.6						
	60	4.34	4.9						
	80	4.58	6.2						
NZ M4	40	5.11	5.0	–2.687	0.456	0.893	0.67	264	1.96
	60	4.68	7.3						
	80	4.87	6.2						

Note:

* The Load Damage Exponent (LDE) described in eqn (1) was calculated from the model coefficients using eqn (4). The method used to determine the LDEs, described in eqns (3), (4) and (5), takes account of the variations in strength noted between some of the experiments.

\# For the observed data fitted to eqn (5).

6. INFLUENCE OF TYRE TYPE AND CONFIGURATION ON PAVEMENT WEAR

A subsequent series of ALF tests was conducted with the wide single and steer tyres shown in Table 2. These tests were conducted using the 40 kN half-axle load and a tyre inflation pressure of 760 kPa (cold). The main variables were tyre configuration (dual or single) and tyre type (wide single or steer tyre). The main indicator of pavement wear monitored during ALF trafficking was surface deformation induced under load. The ALF trafficking resulted in surface deformation levels between 10 and 16 mm for the different tyre configuration after the application of about 150,000 load cycles.

A number of tyre characteristics were recorded to assist in the determination of a relationship between pavement wear and tyre configuration. Pressure-sensitive film was used to obtain information on the individual tyre footprints including the tyre tread contact width, contact length, gross contact area and net contact area (net contact area excludes the gaps between the tyre tread blocks). A series of prints were recorded for each tyre at inflation pressures of 500 kPa, 760 kPa and 900 kPa under the 40 kN half-axle load. Sample tyre-pavement pressure distribution plots for the dual and single tyres are shown in Figure 5.

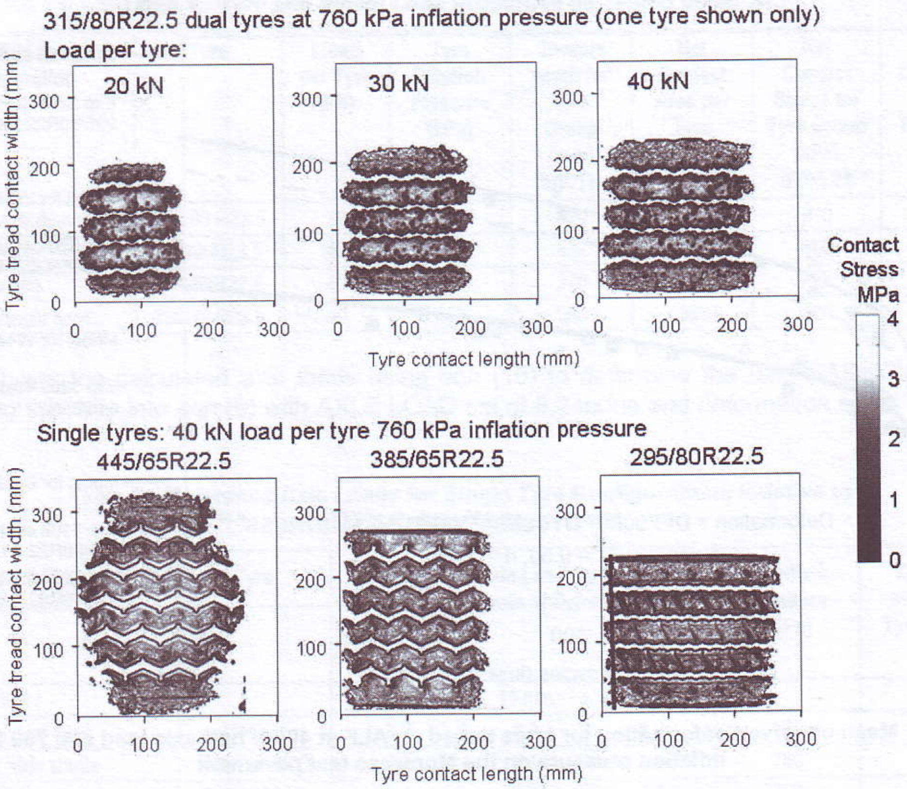


Figure 5: Sample tyre-pavement pressure distribution plots for the dual and single tyres

It was observed that the rate of increase in surface deformation for the wide single tyres and steer tyre was higher than that for the ALF reference dual tyres. A prediction model for the progression of surface deformation was developed based on the ALF data. As with the previous analysis the form of this model involved an initial pavement bedding-in stage covering the first 9,000 load cycles followed by a second stage of deformation progression as shown in eqn (7). In developing the deformation progression model, a multiple linear stepwise regression was undertaken to determine which of the predictor variables were the most significant. From this analysis the most significant predictor variables were found to be the number of cycles of ALF loading (CYCLES), tyre contact width (WIDTH), and net contact stress (STRESS). The net contact stress was determined by dividing the load per tyre by the net contact area per tyre.

A non-linear model was then developed for the prediction of surface deformation progression as shown in eqn (8):

$$\text{Deformation} = \text{DEF9000} + \text{Deformation Progression} \tag{7}$$

where DEF9000 = initial deformation after bedding-in (9,000 load cycles)

and $$\text{Deformation Progression} = \text{CYCLES}^{0.50} \times \text{WIDTH}^{-0.74} \times \text{STRESS}^{0.52} \tag{8}$$

$R^2 = 0.50$; $n = 1194$

The deformation progression model showed a good fit to the observed mean performance data as shown in Figure 6. The model also confirmed the observed trends in that a wider tyre produced a lower level of surface deformation and a higher net stress produced a higher level of surface deformation.

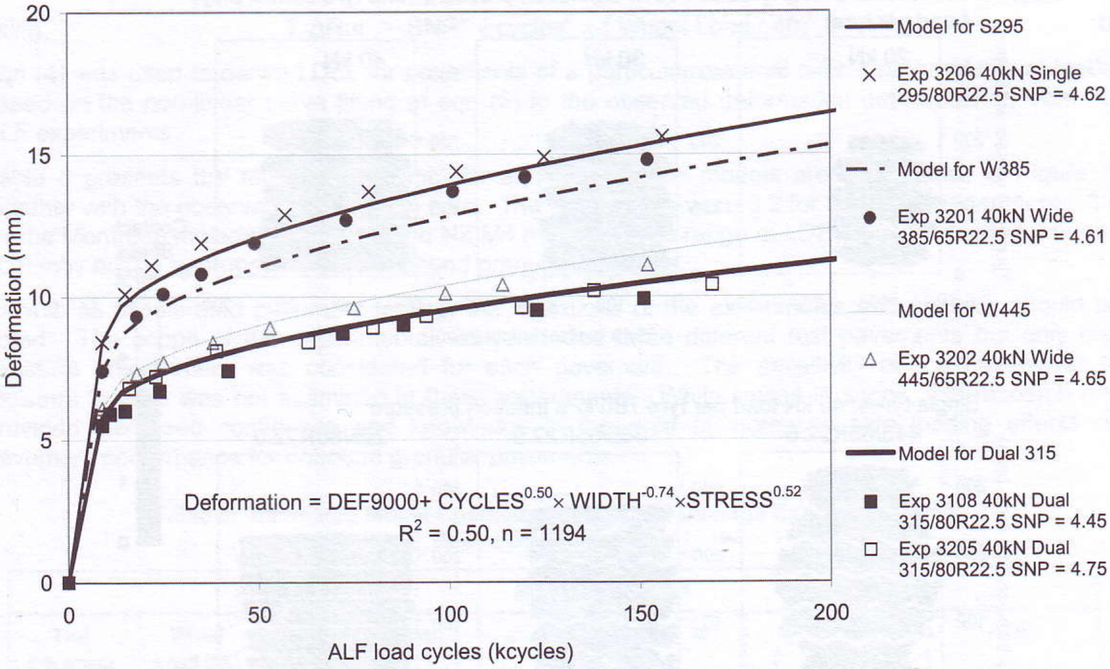


Figure 6: Mean observed deformation for tyres tested on ALF at 40 kN half-axle load and 760 kPa tyre inflation pressure on the Montrose test pavement

6.1 Determination of Single Tyre Axle Loads Relative to Standard Axle with Dual Tyres

Eqn (1) was used to determine the relative axle loads causing the same pavement wear. As discussed above, an LDE value of 3.8 was found to be appropriate for the ALF test pavement; this was rounded to 4 to fall into line with the current Austroads LDE. Eqn (1) was rearranged and the LDE value of 4 was substituted as shown in eqn (9).

$$\text{REFERENCE LOAD} = \text{AXLE LOAD} / [\text{SARs} / \text{CYCLES}]^{(0.25)} \tag{9}$$

For the ALF experiments, the AXLE LOAD was 8.2 tonne represented by an ALF half-axle load on dual tyres of 40 kN.

In order to determine the reference load for the axles having single tyres compared to the Standard Axle load, the ratio SARs / CYCLES was required. The numerator and denominator for this ratio were calculated separately by rearranging eqns (7) and (8) as shown in eqn (10).

$$\text{CYCLES}_A = \frac{\text{Deformation} - \text{DEF9000}_A}{(\text{WIDTH}_A)^{-0.74} \times (\text{STRESS}_A)^{0.52}} \tag{10}$$

where $_A$ represents a particular axle and tyre configuration.

The properties of the axle and tyre combinations tested with ALF, based on the pressure film testing, are shown in Table 4. As the cycles of load derived using eqn (10) are dependent on the extent of deformation, three values were used in the analysis, viz. 10 mm, 15 mm and 20 mm deformation, where the 20 mm deformation represents pavement rutting of about 25 mm which is generally considered to be a terminal pavement condition.

In order to determine the ratio SARs/CYCLES, in addition to the axle/tyre properties listed in Table 4, the properties for the Standard Axle (SAR) were required. Austroads (2004) suggests that a net contact stress of 750 kPa be used for a SAR. Also, it is understood that the most common tyre size in use in Australia is the 11R22.5 tyre and also that this has been adopted on the Standard Axle for pavement analysis. The tread width of a trailer 11R22.5 tyre measured by the pressure film was 190 mm. It was assumed that the initial deformation (DEF9000 in eqn (7)) for a dual tyre group loaded to 20 kN per tyre would be the same as that recorded for the ALF dual tyre experiment at this load.

Table 4: Tyre and Wheel Load Properties as Tested under ALF

Designation and tyre configuration	Tyre	L:oad per Tyre (kN)	Tyre Inflation Pressure (kPa)	Contact width for Tyre Group (mm) WIDTH	Net Contact Area per Tyre (mm^2)	Net Contact Stress for Tyre Group (kPa) STRESS	Initial Deformation after Bedding In (mm) DEF9000
ALF reference dual tyres	315/80 R22.5	20	760	420	44180	910	5.8
wide single tyre	445/65R22.5	40	760	332	46460	860	6.1
wide single tyre	385/65R22.5	40	760	270	39540	1000	7.4
steer axle single tyres	295/80 R22.5	40	760	230	48800	820	8.4

Table 5 shows the calculated axle loads using eqn (10) to determine the ratio SARs/CYCLEs and substituting this ratio into eqn (9) with AXLE LOAD set at 8.2 tonne and deformation set at 10, 15 and 20 mm.

Table 5: Reference Axle Loads for Single Tyre Configurations Relative to 11R22.5 Reference Dual Tyres

Designation and Tyre Configuration on Axle	Tyre	Reference Axle Load (tonne) for Different Levels of Deformation			Tyre Inflation Pressure (KPa)	Contact Width for Tyre Group (mm)
Deformation		10 mm	15 mm	20 mm		
reference dual tyres	11R22.5	8.2	8.2	8.2	560	380
axle with wide single	445/65R22.5	6.9	7.1	7.1	760	332
axle with wide single	385/65R22.5	4.7	5.5	5.7	760	270
steer axle single tyres	295/80 R22.5	3.6	5.1	5.4	760	230

It can be seen from Table 5 that the reference axle load for the steer axle with 295/80R22.5 tyres at 5.4 tonne falls in line with that suggested by Austroads (2004). Interestingly, the reference load on the wider 385/65R22.5 trailing tyre was only slightly greater, at 5.7 tonne.

A further calculation was conducted to determine the load on axles with the three single tyres which would cause equivalent damage to a single axle with dual tyres loaded to the legal limit of 9 tonne. This involved an assumption that the same ratio SARs/CYCLEs would apply for the relative damage at 8.2 tonne compared to 9 tonne on the standard axle. The result was equivalent loads of 7.8 tonne, 6.3 tonne and 6.0 tonne for the 445/65R22.5, 385/65R22.5, and 295/80R22.5 single tyres respectively.

7. SUMMARY

The main findings from the performance data gathered during the experiments addressing the effect of axle load were as follows.

- Surface deformation was highest during the initial bedding-in process, followed by a gradual deformation phase. The rate of deformation was higher for each of the higher axle loads compared to the standard half-axle load of 40 kN.
- The deformation rate (mm/cycle) of the lower quality CC Blend was about 3 to 10 times that of the high quality Montrose material, depending on the magnitude of the wheel load. The deformation rate of the New Zealand M4 material was approximately half that of the Montrose material.
- The stiffness of the test pavements increased, and the deflections decreased, during the bedding-in of the test pavements. This was associated with densification of the base which was greatest at the locations where the initial constructed density was lowest.
- While the differences in performance between different pavement materials were marked, in general the strength (SNP) of all the test pavements was similar.

LDEs for deformation and roughness progression were developed from the performance data based on statistically significant relationships for the prediction of deformation as a function of load cycles, pavement/subgrade strength and axle load. LDEs of 3.2, 3.8 and 2.0 were derived for the CC Blend, Montrose and NZM4 test pavements respectively. These values provided further evidence to support on-going use of the 4th power LDE for granular pavements.

The main findings of the analysis of the performance data addressing the effect of tyre type and configuration were as follows:

- Pavement wear (surface deformation) varied for the different tyre configurations tested. The dual tyres caused less pavement wear than the wide single tyres.
- The relative wear (i.e. the number of cycles to achieve a permanent deformation of 20 mm) was 1.6 times, 3.8 times and 4.7 times for the 445/65R22.5, 385/65R22.5 and 295/80R22.5 respectively compared to the dual tyres.
- The key tyre properties affecting pavement deformation were tyre tread width and net contact stress. A regression model was developed to predict the progression of pavement deformation as a function of these properties.
- The reference load on a steer axle with 295/80R22.5 single tyres which would cause the same pavement wear (vertical surface deformation) as an 8.2 tonne standard axle fitted with dual 11R22.5 tyres was found to be 5.4 tonne. This is in good agreement with recommendations contained in the Austroads (2004) Pavement Design Guide.
- The load on a steer axle fitted with 295/80R22.5 single tyres which caused the same damage as a single axle with dual 11R22.5 tyres at the legal load of 9 tonne was 6.0 tonne, whilst the equivalent axle loads for 385/65R22.5 and 445/65R22.5 single tyres were 6.3 and 7.8 tonne respectively.

While the analysis of the data produced statistically significant models for the prediction of deformation, there were limitations on the outcomes of this research. The research on effects of axle load included only three base materials, each at essentially one moisture content (not subjected to wet and dry periods). Similarly, the tyre configuration research included only one test pavement at one moisture condition. Further work should be conducted on a wider range of pavement types and operating conditions.

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Development of Rapid Construction Methods for Over-Bridge Crossing System*

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ABSTRACT

Serious traffic congestion at intersections has become a critical issue in Japan. Over-bridge crossing systems are very effective in reducing traffic congestion, but sometimes their installation not only creates congestion at other locations owing to traffic restrictions, but also environmental problems are created due to noise, vibration, etc. during construction. As a result, the use of rapid construction has increased because a significant reduction in construction time can reduce economical losses and environmental impacts. The Japan Bridge Association (JBA) has developed a system for the rapid construction of over-bridge crossing systems to minimise the traffic restrictions, reduce construction costs, and minimise environmental impact but at the same time maintaining appropriate quality assurance. An outline of the rapid construction method is presented in this paper, together with two case studies.

1. INTRODUCTION

Serious traffic congestion at intersections has become a critical issue in Japan. According to the Ministry of Land, Infrastructure, and Transport (MLIT), the number of traffic intersection points in Japan which need attention in terms of reducing traffic congestion now exceeds 2,000.

Over-bridge crossing systems are very effective in reducing traffic congestion, but sometimes their installation not only creates congestion at other locations owing to traffic restrictions, but also environmental problems are created due to noise, vibration, etc. during construction. As a result, the use of rapid construction has increased because a significant reduction in construction time can reduce economical losses and environmental impacts.

The Japan Bridge Association (JBA) has developed a system for the rapid construction of over-bridge crossing systems to minimise the traffic restrictions, reduce construction costs, and minimise environmental impact but at the same time maintaining appropriate quality assurance.

An outline of the rapid construction method is presented in this paper, together with two case studies.

2. RAPID CONSTRUCTION METHOD FOR OVER-BRIDGES

When aiming to achieve a significant reduction in construction time, it is important to identify the main causes of construction delays associated with conventional construction methods if these problems are to be addressed. Problems associated with conventional methods, and possible solutions, are shown in Table 1.

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Table 1: Problems and Countermeasures Associated with Conventional Construction Methods

Problems for Conventional Method	Countermeasures
▪ series of construction of superstructures & sub- structures ▪ quantity of site excavated materials (soil and concrete) ▪ time for foundation works	▪ overall construction of superstructures & sub- structures ▪ parallel constructionof approaches ▪ prefabrication of structural members ▪ simple and compact foundation structure ▪ promotion of rapid construction by outcomes

The main characteristics of the rapid construction method developed by the JBA are shown in Table 2. The main advantages of the rapid construction method are as follows.

- ❑ The members used in the superstructure, the substructure and the foundations are mainly made of prefabricated steel.
- ❑ The use of steel results in less load being applied to the foundations; for example, the use of one pile per pier abbreviates the conventional footing as shown in Figure 1.
- ❑ The truck crane staging method can be used for the side span without the any traffic restrictions.
- ❑ A heavy-duty carrier as shown in Fig-2 can be used to minimize traffic disruption when the main spans are being constructed.
- ❑ Parallel construction of the bridge approaches using precast blocks.

Using this method, construction time can be reduced to about 20-30% of the time required if conventional constructuiion methods are applied.

Table 2: Main Characteristics of Rapid Construction Method

Foundation	characteristic	▪ prefabrication of members ▪ simplification of connection between foundation and piers
	type of structure	▪ single pile for single column ▪ direct foundation of steel frames ▪ rigid steel frame footing foundation
	construction method	▪ mobile crane (for steel structures)
Substructure	characteristic	▪ prefabrication of members and application of light-weight structure ▪ seismic performance improved by application of rigid frame structure ▪ no bearing or expansion joints; reduction in maintenance costs
	type of structure	▪ rigid steel frame pier
	construction method	▪ mobile crane
Superstructure	characteristic	▪ prefabrication of members and application of light weight structure ▪ low profile girder with rigid frame structure
	type of structure	▪ steel deck box girder
	construction method	▪ intersection portion: erection of large block using heavy-duty carrier ▪ side spans: staging using mobile crane
Period on Site		3-6 months (incl. approaches); 20-30% of period using conventional methods

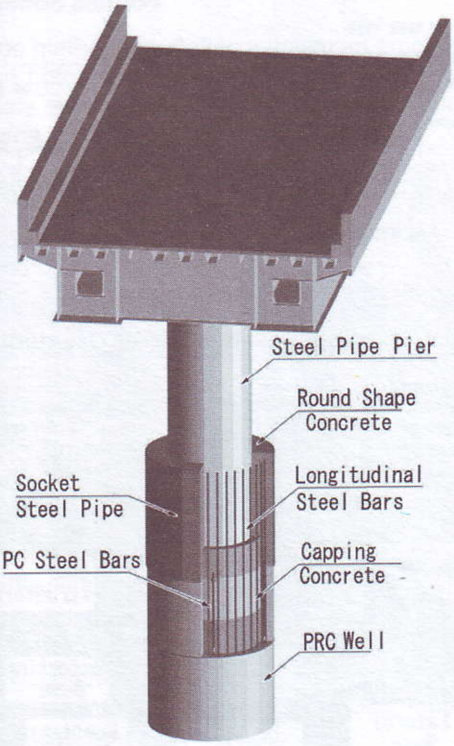

Figure 1: Example of simplified foundation structure

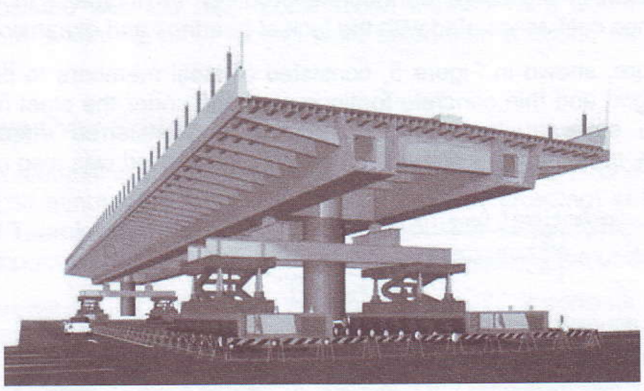
Figure 2: Image of construction of girder using heavy-duty carrier

3. CASE STUDIES

3.1 Harada Viaduct

Harada Viaduct, a four-lane bridge, was designed to mitigate traffic congestion at an intersection in Marugame City, Shikoku. This was the first application of the rapid construction method used by the MLIT. The bridge was tendered in January 2004 and completed in November of the same year.

The general drawing of the Harada Viaduct is shown in Figure 3. The application included seven continuous steel deck box girder rigid frames as the superstructure, rigid frame steel piers as the sub-structure, and compact steel members spread into the foundation. An image of the structure is shown in Figure 4.

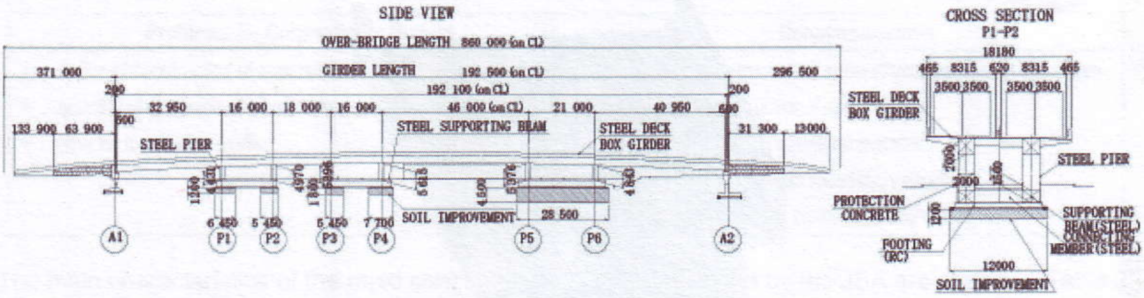

Figure 3: General details of Harada Over-Bridge

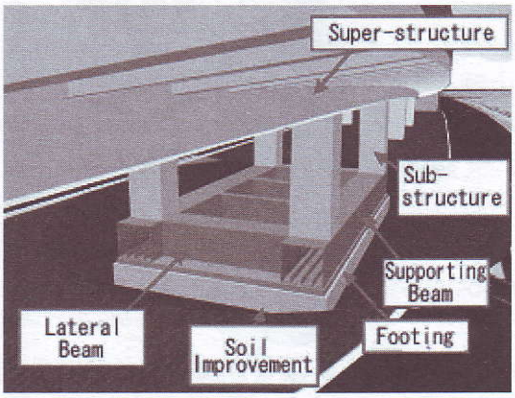

Figure 4: Image of Harada Over-Bridge

Prefabricated steel members were used in the superstructure and substructure to reduce construction time. This also resulted in increased earthquake proofing, an increase in driver comfort, and a reduction in maintenance cost associated with the lack of bearings and expansion joints.

The foundation structure, shown in Figure 5, consisted of steel members to provide support, lateral beams arranged in a grid and thin concrete footings provided under the steel members. In this way, the loading from the superstructure and substructure is transferred effectively into the lower foundation. In addition, the amount of soil which had to be excavated was also reduced.

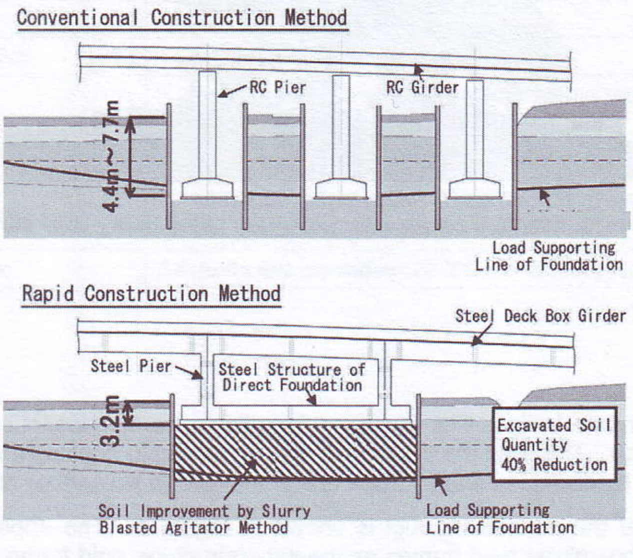

Figure 5: Reduction in soil excavation with soil improvement

3.1.1 *Effect of Rapid Construction Method*

The use of the rapid construction method resulted in a minimum amount of traffic congestion. Figure 6 compares the traffic congestion before and after construction. It is clear from the photographs that traffic congestion was completely resolved and the traffic flow has become much smoother following the construction of the over-bridge.

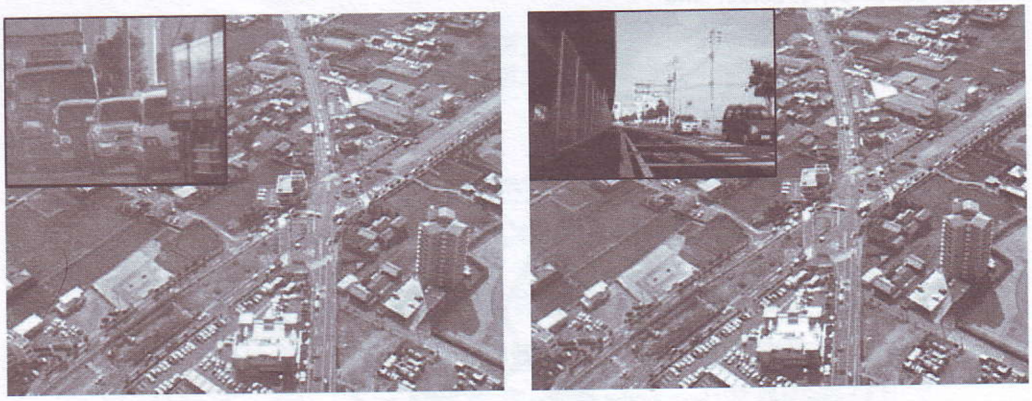

Figure 6: Traffic congestion before and after construction of viaduct

In terms of the construction period, a period of 25 months would be required using the conventional methods compared with only 15 months for the rapid construction method. As far as the over-bridge portion where the rapid construction method was applied, a 60% reduction in total construction time, and a 40% reduction in traffic restrictions, were realised.

Economical Effects due to Congestion Loss

In order to derive the economic benefits associated with the rapid construction method, the MLIT estimated the cost of the time loss due to congestion during traffic restrictions, and the savings associated with the earlier opening of the road to traffic and the resulting smoother traffic flow. The construction cost associated with the rapid construction method were about 20% higher than the conventional method but, when the other factors were taken intio account, an overall cost saving of about 2% was achieved.

3.2 Chofu-Tsurukawa Over-Bridge

The rapid construction method was applied to the Chofu-Tsurukawa over-bridge to mitigate chronic congestion (maximum 34 minutes per hour closing of the railway crossing) at the intersection of the Keio Railway Line and Tsurukawa road in Chofu city, Tokyo. The bridge, which was opened to traffic in March 2003, is a temporary structure until the Keio Line is moved into the underground level.

The general drawing of the bridge structure is shown in Figure 7. It consists of a superstructure of nine spans of continuous steel I-deck rigid frame, and a rigid frame and steel piers substructure. For the spread foundation, precast PC blocks were applied instead of in situ concrete using in the conventional method; precast members were also applied in the bridge approaches.

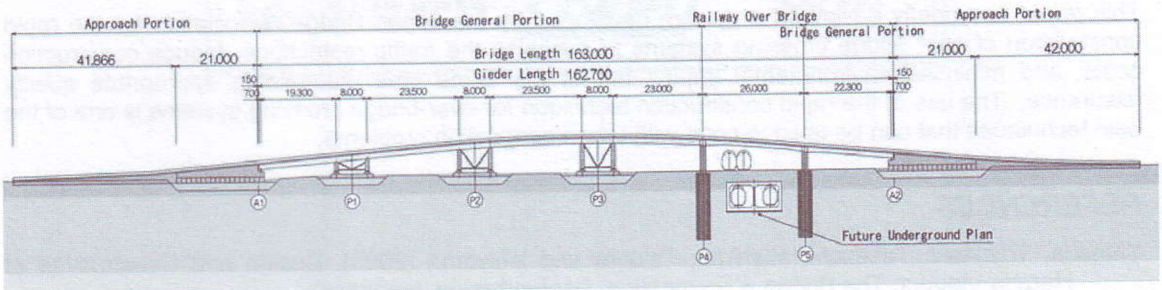

Figure 7: General details of Chofu-Tsurukawa Over-Bridge

The cross section is shown in Figure 8. During the construction of the foundations, several buried pipes were installed to decrease the influence of the upper structures on those pipes and the soil was stabilised to increase the supporting capacity. Rubber bearings were provided in the pier foundation to mitigate the concentration of reactions into the foundation arising from the temperature elongation of the bridge. Precast blocks were adopted for the retaining walls and the deck. Many precast products were used to reduce construction time.

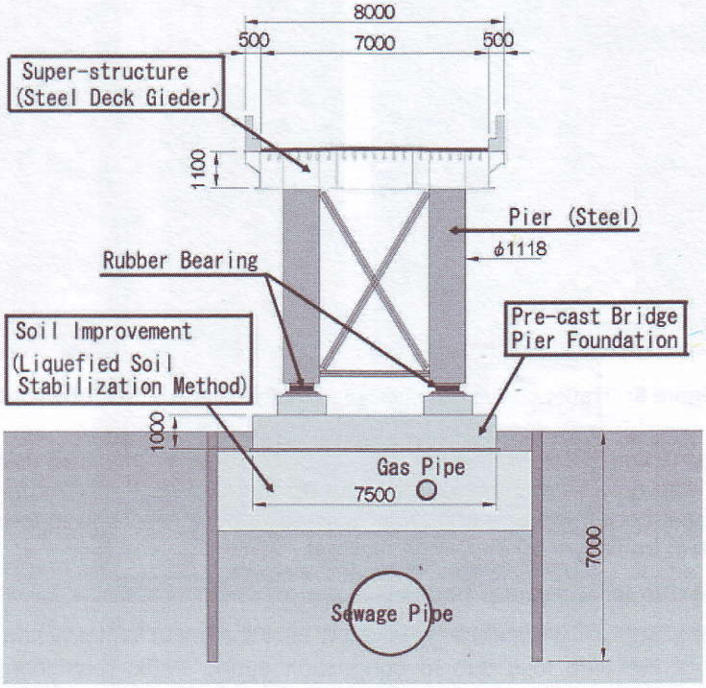

Figure 8: Cross-section of Chofu-Tsurukawa over-bridge

3.2.1 Effect of Rapid Construction Method

The adoption of parallel systems for the construction of the foundation, superstructure and sub-structure resulted in construction being completed in about half the time associated with conventional methods. Investigations conducted by the Tokyo Metropolitan Government office found that:

- passing time was decreased by two-thirds
- the number of cars detouring to nearby roads decreased by about 20%
- the economic benefits (time savings, reduction in expenditure, decrease in traffic accidents, etc.) were about 1000 million yen (US$8.7 million) per year.

4 CONCLUSIONS

This paper has briefly described a system developed by the Japan Bridge Association for the rapid construction of over-bridge crossing systems to minimise the traffic restrictions, reduce construction costs, and minimise environmental impact but at the same time maintaining appropriate quality assurance. The use of the rapid construction technique for over-bridge crossing systems is one of the best techniques that can be used to cope with urban congestion problems.

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